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Seismic Evaluation of the Type L and San Luis Obispo Braced Frame FAA Airport Traffic Control Towers

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Executive Order (EO) 12941 requires all Federal agencies to develop and submit "seismic upgrade" cost estimates to the Federal Emergency Management Agency no later than 1 December 1998. The U.S. Army Corps of Engineers assisted the Federal Aviation Administration (FAA) in responding to this EO by evaluating the seismic resistance of many of their facilities.

This report presents a detailed seismic evaluation of the FAA's Airport Traffic Control Towers in Salinas (shown at right), San Carlos, Palo Alto, and San Luis Obispo, CA.



Foreword

This study was conducted for the Northwestern Division (NWD) of the U.S. Army Corps of Engineers under Military Interdepartmental Purchase Request (MIPR) W81THP80338948, for which the scope of work was defined by the U.S. Army Construction Engineering Research Laboratories (USACERL) proposal "Development of Analytical Procedures and Seismic Evaluation of Selected FAA Airport Traffic Control Towers (ATCTs)." The technical monitor was Bruce H. McCracken, CENWD-NP-ET-E.

This study supported a larger project including other Corps offices, to provide the Federal Aviation Administration (FAA) with the information it needs to formulate a long-range plan to provide seismic upgrades to its most critical facilities. This overall effort is defined in the Interagency Agreement Between the Department of Transportation/Federal Aviation Administration and the US Army Engineer Waterways Experiment Station on behalf of the Corps of Engineers. This work was in direct support of Executive Order (EO) 12941, "Seismic Evaluations of Existing Federally owned or Leased Buildings," dated December 1, 1994; which requires all Federal agencies to develop and submit "seismic upgrade" cost estimates with supporting documentation to the Federal Emergency Management Agency (FEMA) not later than December 1, 1998.

The work was performed by the Engineering Division (FL-E) of the Facilities Technology Laboratory (FL), USACERL. The principal investigator for this project was James Wilcoski. Ernest Heymsfield is an assistant professor for the Department of Civil Engineering at Louisiana State University. Larry M. Windingland is Chief, FL-E, and Michael Golish is Operations Chief, FL. The USACERL technical editors were Gordon L. Cohen and Linda L. Wheatley, Technical Information Team.

Dr. Michael J. O'Connor is the Director of USACERL.

Executive Summary

This report presents a detailed seismic evaluation of the FAA Airport Traffic Control Towers (ATCTs) located in Palo Alto, Salinas, San Carlos and San Luis Obispo, CA. The San Luis Obispo tower is a unique, eccentrically braced steel frame tower. The three other towers are Type L towers, which are reinforced concrete-frame structures. Each was evaluated based on the maximum considered earthquake defined by 1997 National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions (FEMA 302). Type L and the San Luis Obispo towers were evaluated based on several directions of loading and an extreme assumption that the cab windows do not fail and work as fully effective shear walls.

Type L ATCTs

The San Carlos ATCT was the most critical Type L tower, due to excessive deflections in the tower cab. These deflections were due to large rotations of supporting members at the shaft roof. The cab columns (corner mullions) connection base plates were also overstressed as indicated by very large demand capacity ratios. Hinges would form at the base of each mullion due to base plate bending failure, causing a collapse mechanism at very low seismic motions.

An upgrade approach was developed and demonstrated that reduces deflections to acceptable levels and protects the vulnerable connections. This upgrade consists of welding deep structural tubing members to the base of each corner mullion in a pentagon configuration as shown in Figure 11 of the report. The mullions themselves were also stiffened and strengthened by welding 5" x 1.5" plates on both faces of the mullions.

San Luis Obispo ATCT

The shaft braces were the most critically stressed components in the San Luis Obispo tower shaft. These would buckle at several floor levels. However, when the braces were assumed to be tension-only members, they had adequate capac-

ity. These braces could yield slightly, but this would be very limited and deflections would be kept within acceptable levels.

Deflection in the tower cab could be large, but within acceptable levels. The most vulnerable cab component is the connection of the corner mullions to their base plates. This vulnerability is due to shear failure of the fillet welds at this connection. However, serious damage to these connections should be prevented by redistribution of forces to other mullions and other building components. Therefore, the San Luis Obispo tower passed this evaluation by meeting the life-safety requirements.

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1 Introduction

Background

This work was conducted in response to Executive Order (EO) 12941, which promulgated the National Earthquake Hazards Reduction Program (NEHRP) Act, Public Law 101-614. EO 12941, “Seismic Evaluations of Existing Federally owned or Leased Buildings,” dated December 1, 1994, requires all Federal agencies to develop and submit “seismic upgrade” cost estimates with supporting documentation to the Federal Emergency Management Agency (FEMA) not later than 1 December 1998.

The U.S. Army Corps of Engineers assisted the Federal Aviation Administration (FAA) in responding to this EO by evaluating the seismic resistance of many of their facilities. The Northwestern Division (NWD), Waterways Experiment Station (WES), and Construction Engineering Research Laboratories (USACERL) worked together to deliver these evaluations. The first phase of the evaluations included structural evaluation and guidance for nonstructural evaluation of FAA airport traffic control towers (ATCTs). The Corps provided detailed retrofit guidance and cost estimates for seismically vulnerable ATCTs. These evaluations were completed in 1996 for standard control towers in Seismic Zones 3 and 4.

The second phase of this work began in 1997, when the Corps evaluated numerous other FAA facilities.

Objective

The objective of this project was to conduct life-safety seismic evaluations of the FAA ATCTs at the California locations shown in Table 1. These structural evaluations included both the tower shaft and cabs. Retrofit schemes were also developed for towers found to be vulnerable.

Approach

This report presents the evaluation of the Type L concrete frame towers and a braced steel frame tower in Seismic Zone 4. Table 1 is a summary of the Type L and San Luis Obispo steel braced frame ATCTs taken from the URS Greiner final report.*

Figure 1 shows the Salinas tower, which will have the most critically loaded shaft of all the Type L towers because it has the greatest height (50 ft). Some interior members in the shorter San Carlos tower (Figure 2) may be loaded more severely than Salinas, because of the higher spectral acceleration levels. Therefore, selected members and connections will be evaluated based on the analysis of the shorter San Carlos tower. The Palo Alto and San Carlos towers are structurally identical, except that the San Carlos tower experiences greater seismic loads. Retrofit schemes derived for the San Carlos tower, therefore, would be sufficient for the Palo Alto tower.

The San Luis Obispo tower is a unique tower with a steel braced frame shaft and steel moment frame cab. Figure 3 shows the San Luis Obispo tower. All towers were evaluated for both gravity and seismic loading using SAP 2000 Finite Element Method software.†

Table 1. Summary of Type L and San Luis Obispo ATCTs.

California Location	Height at Cab Base (ft)	Year Built	UBC Seismic Zone	FEMA 302 Short Period Spectral Acceleration, S_{ps}	Construction Type	
					Shaft	Cab
Salinas (SNS)	50	1968	4	1.00 g	Reinforced concrete frame	Steel moment frame
San Carlos (SQL)	30	1969	4	1.07 g	Reinforced concrete frame	Steel moment frame
Palo Alto (PAQ)	30	1965	4	1.00 g	Reinforced concrete frame	Steel moment frame
San Luis Obispo (SBP)	45	1988	4	1.00 g	Steel braced frame	Steel moment frame

* URS Greiner, Federal Aviation Administration Categorization of Airport Traffic Control Towers (ATCTs) Seismic Screening of Terminal Radar Approach Control (TRACON) Buildings, for the U.S. Army Corps of Engineers, 20 May 1998.

† SAP2000 Structural Analysis Programs from Computers and Structures, Inc., Berkeley, CA.

The ATCTs shown in Table 1 were evaluated following the steps presented in Chapter 3, **Analysis Steps and Cases**.

Mode of Technology Transfer

This report will be incorporated into the overall Corps of Engineers effort to provide the FAA with the information it needs to formulate a long-range plan to provide seismic upgrades to its most critical facilities. This report will also contribute to the FAA response to EO 12941 requiring “Seismic Evaluations of Existing Federally Owned and Leased Buildings.”



Figure 1. Salinas, CA, Type L ATCT, 50-ft tall.



Figure 2. San Carlos, CA, Type L ATCT, 30-ft tall.



Figure 3. San Luis Obispo, CA, steel braced frame ATCT, 45-ft tall.

References

The following references were used in these evaluations:

1991 Edition of the NEHRP Recommended Provisions and Commentary for the Development of Seismic Regulations for New Buildings (FEMA 222 and 223), January 1992.

1997 Edition of the NEHRP Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures (FEMA 302 and 303), February 1998.

1997 Uniform Building Code, Volume 2: Structural Engineering Design Provisions.

AISC Manual of Steel Construction, Load & Resistance Factor Design (LRFD), 2nd Edition, 1994.

Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95), American Concrete Institute, October 1995.

Corlett & Spackman, Drawings of Air Traffic Control Towers, San Carlos and Salinas, CA, (Corlett & Spackman, Architects, San Francisco, CA, June 1966).

Leo A Daly, Drawings of Administrative Base Building, San Luis Obispo, CA (Leo A Daly Architecture/Engineering, Omaha, NE, December 1981).

National Earthquake Hazards Reduction Program (NEHRP) Guidelines (FEMA 273) and Commentary (FEMA 274) for the Seismic Rehabilitation of Buildings, October 1997.

Units of Weight and Measure

U.S. standard units of measure are used throughout this report. A table of conversion factors for Standard International (SI) units is provided below.

SI conversion factors		
1 in.	=	2.54 cm
1 ft	=	0.305 m
1 kip force	=	4.45 kN
1 k-in.	=	113 N-m
1 ksi	=	6.89 MPa
1 lb mass	=	0.453 kg
1 lb/ft	=	1.488 kg/m
1 pcf	=	16.02 kg.m ³
1 psf	=	4.882 kg/m ²

2 Tower Configurations, Model Assumptions, and Load Conditions

The analytical approach used included gravity and response spectrum analysis, first using a linear dynamic procedure (LDP; FEMA 273, 3.3.2) and then if needed using a nonlinear static procedure (NSP), i.e., “pushover analysis” (FEMA 273, 3.3.3). If the LDP indicates significant inelastic demand in a tower shaft, that tower will be evaluated using the NSP. This inelastic demand will be quantified in terms of demand-capacity ratios (DCRs), from the LDP (FEMA 273, 2.9.1.1). If any DCRs exceed 2.0, except for cab elements, the tower may be evaluated using an NSP. DCRs in excess of 2.0 were expected in the tower cabs, particularly at the window mullions. Significant inelastic response of the cab should not have much influence on the performance of the tower shaft. This is because of the small weight of the cabs relative to that of the shafts. Though the cabs provide a very critical function, they are similar to a penthouse appendage. More detailed evaluation of the cabs was performed if several cab members or connections had high DCRs.

Tower Foundation Assumptions

The tower foundations were modeled as fixed bases (FEMA 273, 3.2.2.6; FEMA 302, 5.4.4). Rotation at the base of the structure would cause small increases in moments due to P-delta effects. However, these structures are relatively light-weight and P-delta increases in loads should be negligible, so this effect was neglected. Neglecting base rotations also decreases calculated total deflections. In this evaluation, relative (not absolute) displacements were most critical for evaluating tower vulnerability.

Type L Tower Shaft Configuration and Model

The Salinas tower shaft is a 50-ft tall reinforced concrete frame and is square in plan. The tower shaft consists of four tapered bents (one at each corner) joined at the center of the top of the shaft. These column bents protrude out diagonally from the shaft corners, and taper from a maximum width of 82 in. at ground

level to a minimum width of 36 in. at the top of the shaft (Drawing S1; Corlett & Spackman 1966). The vertical portion of three bents can be seen in Figure 1. The horizontal portion of each concrete bent is 24-in. deep. The bents are 18-in. wide in both the vertical column and horizontal beam portions. The reinforced concrete floor slabs are supported by steel beams, which in turn frame into the concrete bents. These beams are constructed with shear studs so that they will behave as a composite section with the floor slabs. The floor slabs are 5-in. thick, consist of one-way construction, with two layers of number 4 bars at 12 in. on center, and span approximately 7 ft to the beams. Shear transfer from slab to beam is accomplished using $\frac{3}{4}$ -in. welded studs at 12 in. on center along the beam centerline. Beams are typically 16B26 and 16WF40 (similar to currently available W16x26 and W16x40) and they span 21 ft to the four corner columns. The column bents are supported by reinforced concrete grade beams, which act both as spread footings and tie elements. Additionally, tie elements are provided along the diagonal of the structure, at the foundation, to prevent the columns from spreading apart under gravity loads.

The tower lateral-load-resisting system consists of concrete floor diaphragms that bear upon reinforced concrete moment frames. The frames are formed by the four corner column bents, which create two individual frames, one in each orthogonal direction at the diagonal of the shaft plan. These columns extend to the top of the tower shaft. The column bents are reinforced with three number 14 bars at each face (strong axis of columns) up to an elevation of 24 ft 6 in., plus three additional number 14 bars on each face up to an elevation of 12 ft 6 in. Four number 5 bars are also located on each face (weak axis of columns) that fan out at equal spacing over the entire elevation of the columns. Three number 11 bars at each face (strong axis) begin below the 24 ft 6 in. elevation and are spliced to the number 14 bars. Column shear reinforcement consists of number 4 bar ties at 9 in. on center for the bottom 25 ft and number 4 bars at 12 in. on center for the remaining column elevation. The horizontal beam portion of the bents are reinforced with three number 10 bars at the top and bottom of the beam. Additional reinforcement is provided at the corner of the bent. Shear reinforcement of the horizontal beam portion of the bent consists of number 3 bar ties at 12 in. on center.

Each floor level is connected at an intermediate point of the frame columns (bents). Lateral loads are transferred from the floor diaphragms to the columns by either compression bearing or by tension in embedded anchor bolts. The two moment frames cross each other at the center of the shaft roof in plan. A hinge is installed in each moment frame at this location that decouples the frames in the two orthogonal directions, allowing the frames to act independently. If the

hinge were not present, large torsional forces would be applied to the frame elements during lateral seismic motions.

The reinforced concrete bents were modeled using beam elements. The full gross cross-sectional area of the concrete was used in these elements, as this will produce the largest forces and moments. The pinned bent connection at the top of the shaft was modeled as a pinned connection in all three directions. The concrete floors and wide flange beams at each floor level are modeled as rigid diaphragms. The concrete floor slabs are designed to behave composite with the wide flange beams and the section properties used to model these elements use the effective width of the concrete slab.* The connections of the floor wide-flange beams to the bents are modeled as pinned connections in both principal axes of the beams. The shaft roof does not have a concrete slab, so this is not modeled as a rigid diaphragm. Figure 4 shows the finite element mesh that represents the shaft and cab of the 50 foot tall Salinas ATCT.

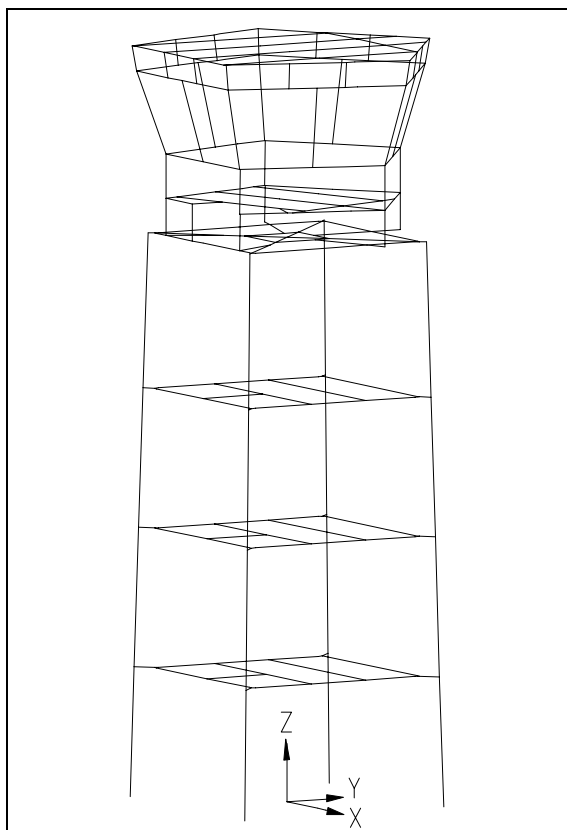


Figure 4. Finite element mesh for the 50-ft tall Salinas tower.

* AISC LRFD, I3.1.

Gravity and seismic loads are applied to the model by including the self-weight in appropriate structural elements and adding the effects of seismic loads through the use of a response spectrum. All self-weight beyond that in the structural members is added to the model by adding to the unit weight of beam members of the structure. Normally 50 percent of this self-weight was added to the beams at the perimeter of the structure and the other 50 percent to interior beams. The weight of wall cladding and stairs was distributed equally between the floor below and above the floor. The weight of interior partition walls and doors was distributed with one-third to the floor above and two-thirds to the floor below. Appendix A provides information on the calculation of weights distributed to each member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 2 for the Salinas tower.

The effects of horizontal torsion were considered by offsetting the center of mass of both self-weight and live loads 5 percent of the floor width in the direction perpendicular to the primary (100 percent response spectrum) lateral loading. The direction of mass offset is 180 degrees from the maximum lateral stiffness, so that the maximum distance, perpendicular to the primary load, is created between the center of stiffness and of mass (FEMA 273, 3.2.2.2). This offset of 5 percent was obtained by factoring the member weight and mass as shown in Appendix A. These factors apply only to the distributed floor weight, not the member self-weight or cladding weight. The gravity loads are applied in this manner for all floor levels. The actual loads placed on each member are summarized in Appendix A.

Table 2. Salinas Type L Tower Distributed Loads.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
2nd	2662	2828	22171	15	93	195
3rd	2515	2828	22171	15	93	194
4th	3447	2761	23325	15	95	41
Top of Shaft	1304	2096	14824	5	43	94
Cab Floor	329	133	17796	5	51	33
Cab Roof	-	-	15071	5	25	13

Type L Tower Cab Configuration and Model

The tower cab is configured as a pentagon in plan. The lateral load resisting system of the tower cab consists of light structural steel moment frames. The cab columns are built-up structural tubes, which are 7-in. deep and 4-in. wide. The columns also serve as the window corner mullions. These will likely be the most

vulnerable members in the Type L towers. The cab floor is a steel deck with a cast-in-place concrete slab, supported with wide flange beams. The cab framing is connected to the shaft roof framing, which in turn is connected to the concrete frames by embedded anchor bolts. The cab roof is a steel deck supported by steel beams, which frame into the supporting cab columns (window mullions). The top and bottom of the windows are supported with channel sections that are oriented with their strong axis perpendicular to the plane of the window. At the center of each face of the cab, the windows are divided and supported by smaller interior mullions (S3 x 7.5).

The tower cab was modeled using beam elements for the mullions and beams. Mullion/beam connections are fixed at the base of the mullions only. Rigid diaphragms connect all members at the cab floor and roof levels, preventing relative horizontal deflection, while allowing vertical flexure. The cab roof and floor are both modeled as rigid diaphragms, even though only the cab floor has a concrete slab. The rigid diaphragm at the roof is slightly unconservative for members in the roof, because it will eliminate weak axis bending and axial loading on the beam elements at this level. However, these loads would be insignificant, plus the steel roof deck will in reality provide a degree of rigidity, especially in the weak axis direction of the beams, which is the direction of roof deck corrugation. The rigid roof diaphragm will more realistically provide the model performance that will more heavily load critical cab members.

Cab windows have often failed in past earthquakes because stress concentrations form where the brittle glass comes in contact with the frame. However, if the glass does not fail, and it effectively works with the window mullions and other window frame members, it will perform as a stiff shear wall. This condition will load the cab members below the window more severely, plus it will load the shaft more heavily. The stiffness of the cab will increase significantly, so that the natural period decreases, which will couple the first mode of the cab with the first mode of the shaft. This decrease in period will also increase the effective acceleration of the cab due to the shape of the response spectra. Therefore, the worst case shaft evaluation will be the case where all the cab windows act as fully effective shear walls. This is modeled by increasing the moment of inertia of the members around the perimeter of the window to equal the shear stiffness of the windows.

The 30-ft tall San Carlos tower will be more severely loaded at the cab. The cab of this tower is identical to the Salinas tower. The San Carlos cab will be more severely loaded because of the shorter tower height making it stiffer with a smaller natural period and higher effective acceleration. The cab will also be more severely loaded due to the higher spectral acceleration values at San Car-

los. In particular, the San Carlos spectrum is greater at higher periods that correspond to the natural period of the cabs. Therefore, Type L cab evaluation will be based on the San Carlos tower. Figure 5 shows the finite element mesh that represents the shaft and cab of the 30-ft tall San Carlos ATCT. Figure 6 shows the model for the cab by itself. This figure shows the shape and orientation of each structural member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 3 for the San Carlos tower.

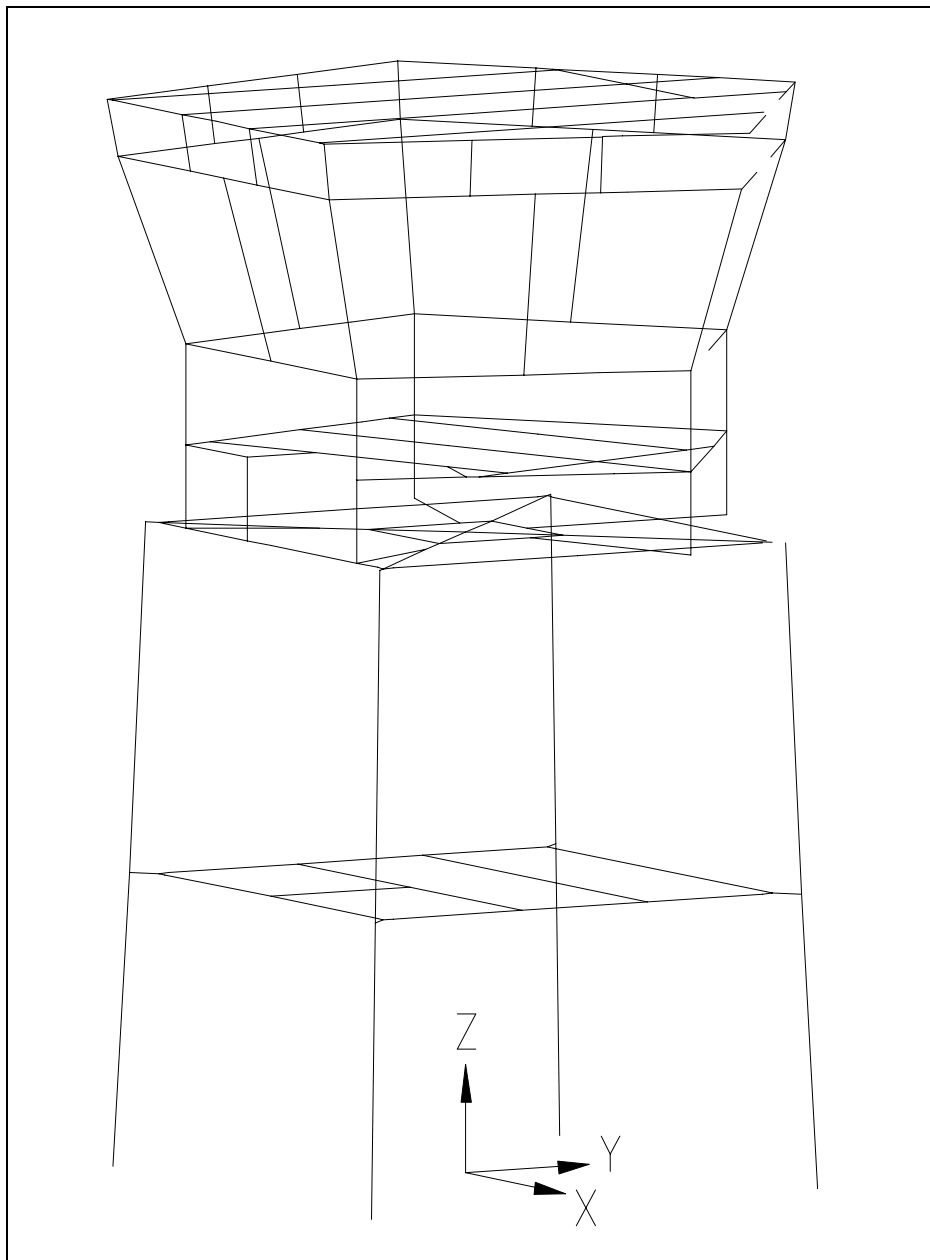


Figure 5. Finite element mesh for the 30-ft tall San Carlos tower.

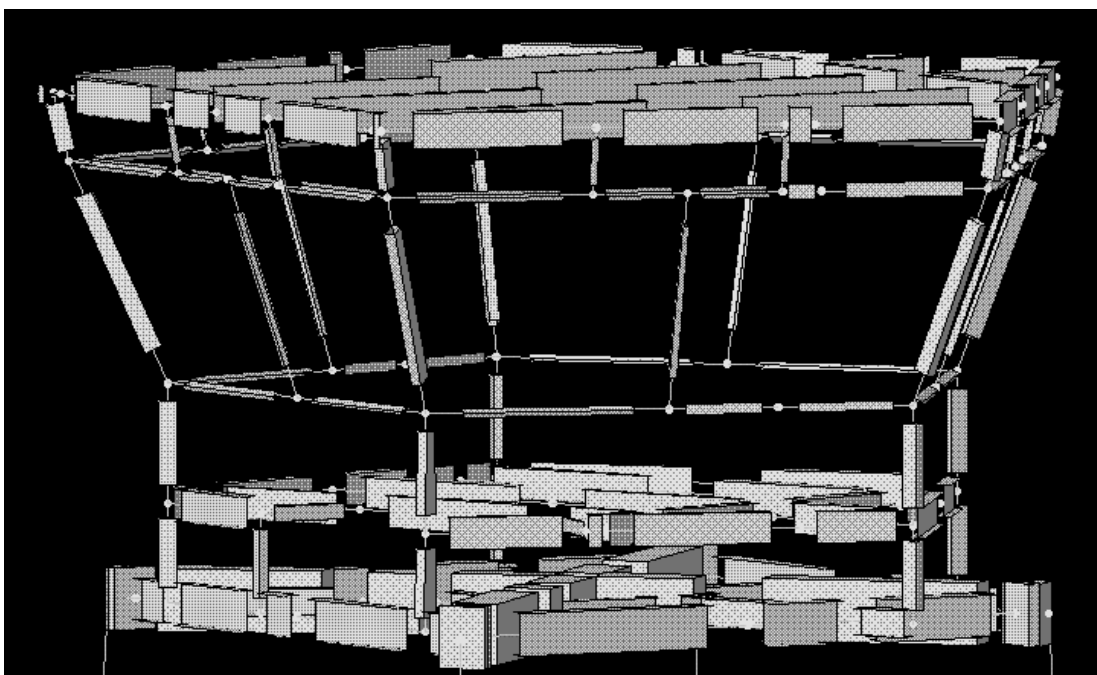


Figure 6. Finite element mesh for the cab of the San Carlos tower.

Table 3. San Carlos Type L Tower Distributed Loads.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
2nd	3602	2761	23325	15	96	191
Top of Shaft	1304	2096	14824	5	46	94
Cab Floor	329	133	17796	5	51	33
Cab Roof	-	-	15071	5	25	13

Loads were distributed along the horizontal beam members at the cab floor, channels above and below the windows, mullions, and roof. The cab gravity loads are calculated as shown in Appendix A, and summarized for the cab floor and roof in Table 3. The seismic loads are applied using the response spectra as in the tower structure. As with the shaft, the weights and masses are distributed so as to provide 5 percent eccentricity of the distributed weight at the cab floor and roof levels. The weight and mass of exterior cladding and window glass are assumed to be uniformly distributed around the building perimeter without any offset (eccentricity). The cab window loads can be summarized as follows:

- Plate glass density = 161 pcf*

* 1994 AISC LRFD, Table 7-4, Weights and Specific Gravities, p 7-6.

- Single 3/8 in.-thick pane
- $161 \text{ pcf} \times 0.375 \text{ in./12 in./ft} = 5.03 \text{ psf}$
- Total weight per window pane = $5.03 \text{ psf} \times 56.23 \text{ sf} = 283 \text{ lb}$
- Distribute this weight equally to the members around the window perimeter, $283 \text{ lb}/398 \text{ in.} = 0.711 \text{ lb/in.}$

San Luis Obispo Tower Shaft Configuration and Model

The San Luis Obispo tower was built in 1988. The tower is 45 ft tall at the cab base, 48 ft at the cab floor and 64 ft at the top of the parapet. The shaft is square in plan with its sides measuring 20 ft in length. The tower construction consists of structural steel framing covered with insulated metal panel siding.

The vertical load resisting system consists of concrete-topped metal decking supported by wide flange steel beams and columns that bear on concrete foundations. Typical floor framing consists of 1-1/2-in. deep, 20 gauge metal decking, topped with 2-1/2-in. of concrete fill. The floor slab spans as much as 7 ft to the beams. Interior beams are W12 x 30, and the supporting beams around the perimeter that frame into the columns are W14 x 34, W16 x 40, and W16 x 45. The floor supporting steel beams do not include shear studs, so that the concrete is not designed to act as a composite section with the beams. The columns are W8 x 31 for the lower 23 ft of the tower shaft and W8 x 24 to the top of the shaft. The foundation consists of 6 ft x 6 ft x 18-in. deep spread footings under each column.

The tower lateral load resisting system consists of an eccentric braced steel frame, located symmetrically around the tower perimeter. These braces are L6 x 6 x 1/2 single angles with bolted connections. These braces frame into the beams so that the center 4-ft 3-in. portion of the beam is loaded eccentrically. These portions of the beam (a shear link) will carry large shear and moment loads and is intended dissipate energy in an earthquake. Additional W8 x 21 stub columns are added to the story below the shaft roof, below the base of the cab mullions. These work with the braces and beam below to form a partial truss for carrying gravity and seismic loads from the cab.

All structural members were modeled using beam/column frame elements. The concrete floors at each floor were modeled as rigid diaphragms. The shaft roof, however, does not have a concrete slab or even a roof deck, because the cab and

walkway around the cab perimeter cover the entire shaft. All connections between the columns, beams, and braces in the shaft were modeled as pinned. Figure 7 shows the finite element mesh for both the shaft and cab of the San Luis Obispo steel braced frame tower.

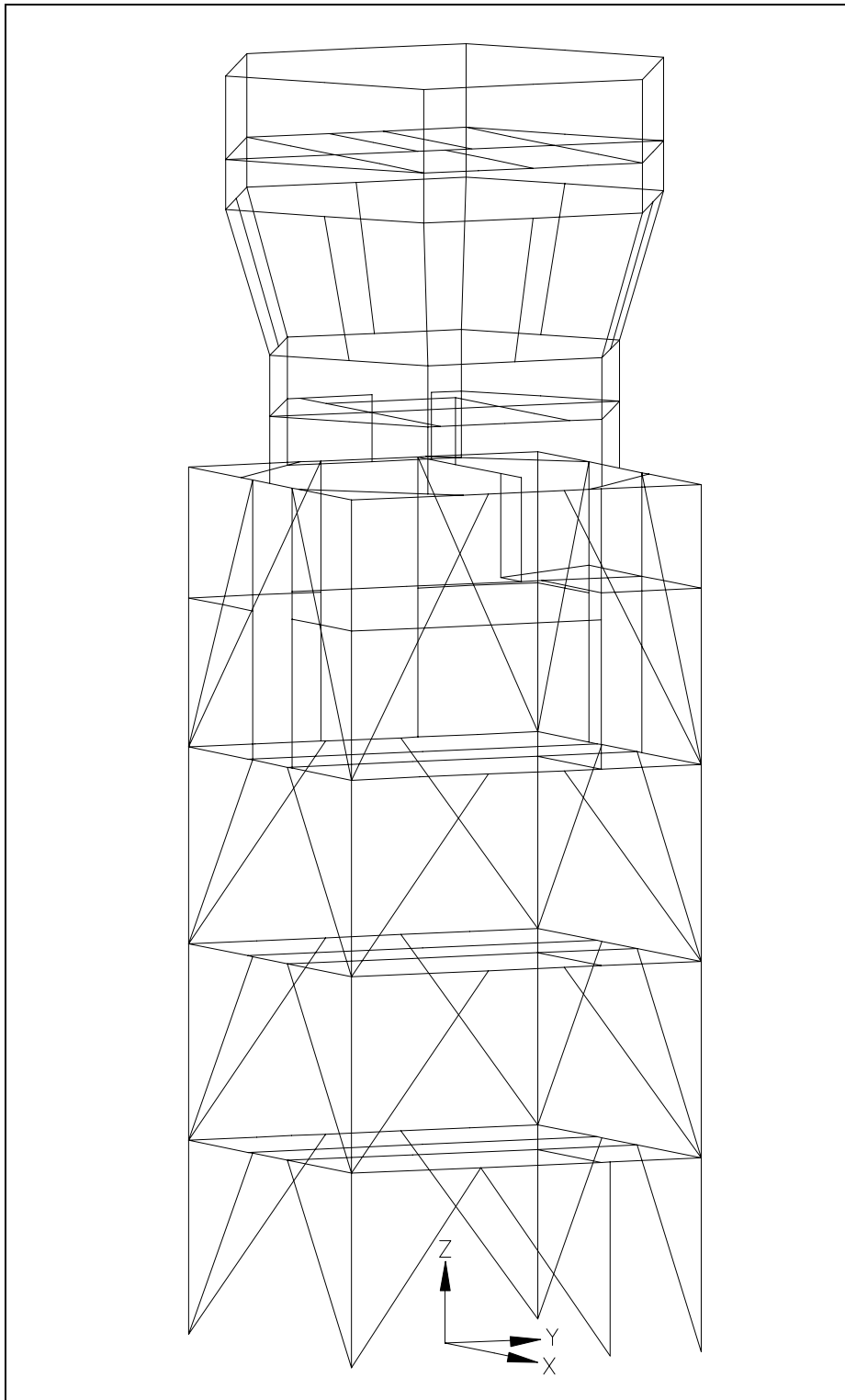


Figure 7. Finite element mesh for the 45-ft tall San Luis Obispo tower.

Gravity and seismic loads were applied to the model by including the self-weight in appropriate structural elements and adding the effects of seismic loads through the use of a response spectrum. All self-weight beyond that in the structural members was added to the model by adding to the unit weight of beam members of the structure. Normally 50 percent of this self-weight was added to the beams at the perimeter of the structure and the other 50 percent to interior beams. At the top of the shaft, 75 percent of the self-weight beyond the members was added to the perimeter beams and the remaining 25 percent to the interior beams. This level does not have a floor slab, so that most of the load at this level is near the building perimeter (from the walkway and cladding).

The weight of wall cladding, partition walls, and doors was distributed with one-third to the floor above and two-thirds to the floor below. The wall cladding distribution differs from the Type L distribution, which was distributed equally to the floor above and below. The weight of stairs was distributed equally between the floor above and floor below. Appendix C provides information on the calculation of weights distributed to each member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 4 for the San Luis Obispo tower.

As with the Type L towers, the effects of horizontal torsion were considered by offsetting the center of mass of both self-weight and live loads 5 percent of the floor width in the direction perpendicular to the primary lateral loading. The actual loads placed on each member to achieve this eccentricity are summarized in Appendix C.

Table 4. San Luis Obispo Distributed Loads*.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
Intermediate 2	8200	4282	11941	15	77	36
Intermediate 3	7663	4282	11941	15	75	36
Junction	11621	4467	11690	15	89	47
Cab Access	2890	2541	3113	15	25	0
Top of Shaft	6657	1103	18809	5	75	27
Cab Floor	1062	193	10339	5	60	109
Cab Roof	0	0	5687	5	21	60

* The top of shaft floor system weight includes the walkway around the cab perimeter. The cab floor and cab roof cladding weights include the weight of the glass and the parapet cladding respectively. The areas of openings are included in areas used in calculating the "Total at Floor (psf)."

San Luis Obispo Tower Cab Configuration and Model

The San Luis Obispo tower cab is configured as a hexagon in plan. The lateral load resisting system of the tower cab consists of light structural steel moment frames. The cab columns are made from TS 8 x 4 x ½ structural tubing members. The columns also serve as the window corner mullions. These will likely be the most vulnerable members in the San Luis Obispo towers. The cab columns are welded to base plates, which are in turn welded to beams at the top of the shaft. The welded connection from this base plate to the beams is critical and details on the drawings are incomplete. Field inspection revealed that the base plate is welded around its entire perimeter to the supporting beam flange.* The cab floor is a steel deck with a cast-in-place concrete slab, supported with wide flange beams. These beams are connected to the cab columns with shear connections. The cab roof is a steel deck supported by wide flange beams, which frame into the supporting cab columns (window mullions), using shear connections. Horizontal structural tubing members (TS 7 x 7 x 3/16) span the cab columns above and below the windows and support the windows. These horizontal tubes are connected to the columns with full penetration groove welds all around the tube, making these moment connections. At the center of each face of the cab, the windows are divided and supported by smaller interior mullions made of TS 4 x 2 x 3/16 structural tubes. These interior mullions are welded† to supporting TS 7 x 7 x 3/16 members at both their tops and bottoms, making these moment connections. Above the cab roof, another horizontal TS 7 x 7 x 3/16 structural tube member frames into the cab columns at the top of the parapet wall. These members are also welded to the cab columns with a full penetration groove weld forming a moment connection.

The tower cab was modeled using beam/column frame elements for the columns and beams. Column/beam connections are fixed, at their base, bottom and top of windows and at the top of the cab parapet. Connections between the column and beams at the cab floor and roof levels are modeled as pinned because they are shear connections. Rigid diaphragms connect all members at the cab floor and roof levels, preventing relative horizontal deflection, while allowing vertical flexure. The cab roof and floor are both modeled as rigid diaphragms, even though only the cab floor has a concrete slab. The rigid diaphragm at the roof is slightly unconservative for members in the roof, because it will eliminate weak axis

* Inspected by Gary Benson (FAA representative at San Luis Obispo) on 11 August 1998.

† Based on field inspection.

bending and axial loading on the beam elements at this level. However, these loads would be insignificant, plus the steel roof deck will in reality provide a degree of rigidity, especially in the weak axis direction of the beams, which is the direction of roof deck corrugation. The rigid roof diaphragm will more realistically provide the model performance that will more heavily load critical cab members.

As explained in the Type L tower cab model assumptions, the worst case shaft evaluation will be the case where all the cab windows act as fully effective shear walls. This is modeled by increasing the moment of inertia of the members around the perimeter of the window to equal the shear stiffness of the windows. Figure 8 shows the San Luis Obispo cab model, including members at the shaft roof level and above. This figure shows the shape and orientation of each structural member.

Loads were distributed along the horizontal beam members at the cab floor, tubing above and below the windows, and roof and tubing at the top of the parapet. The cab gravity loads are calculated as shown in Appendix C and summarized for the cab floor and roof in Table 4. The seismic loads are applied using the response spectra as in the tower structure. As with the shaft, the weights and masses are distributed to provide 5 percent eccentricity of the distributed weight at the cab floor and roof levels. The weight and mass of exterior cladding and window glass are assumed to be uniformly distributed around the building perimeter without any eccentricity. The cab window loads can be summarized as follows*:

- Glass weight (1-in. thick pane) = 15 psf[†] (180 pcf)
- Total weight per window pane = 15 psf x 5.4 ft x 7.8 ft = 634 lb
- Distribute one-third and two-thirds of this weight to the TS 7 x 7 x 3/16 tubes above and below the windows, respectively.

* Window weight calculations differed slightly from the Type L towers as shown here.

† 1994 AISC LRFD, Table 7-5, Weights of Building Materials, p 7-7.

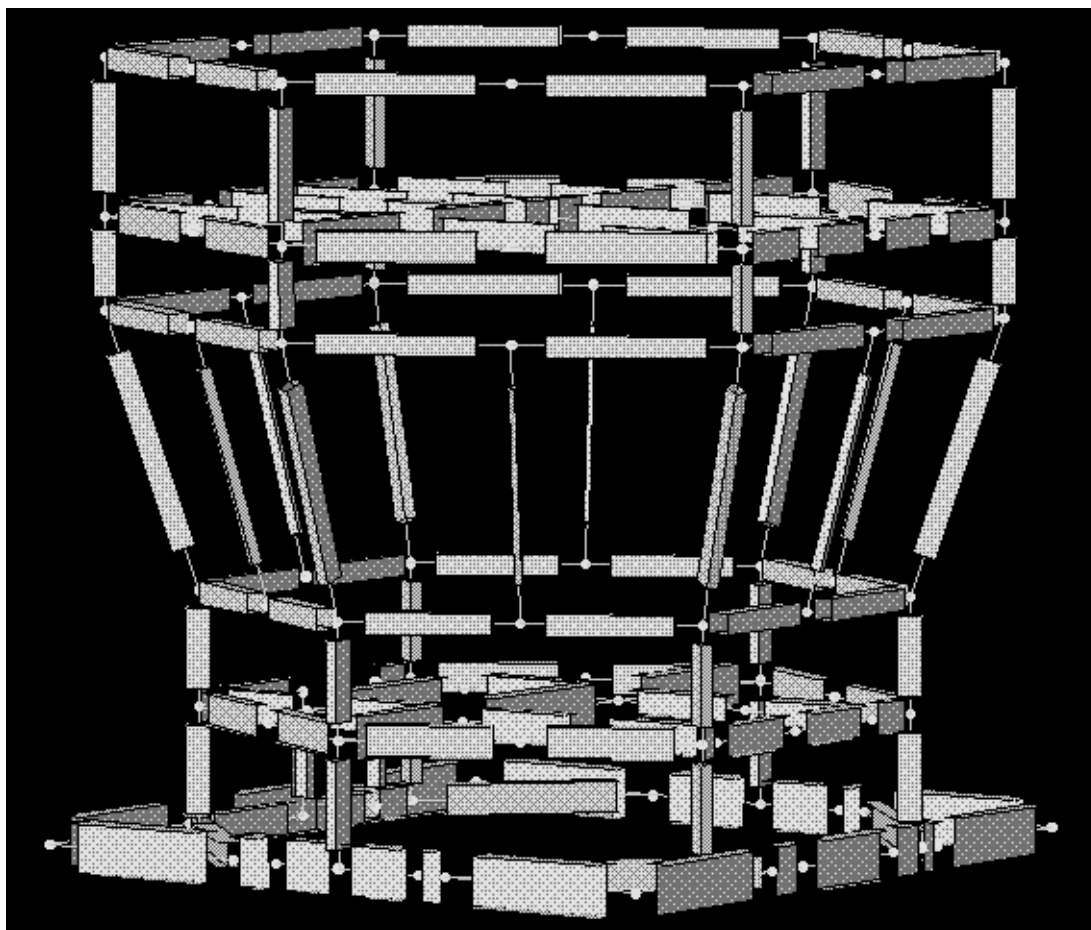


Figure 8. Finite element mesh for the cab of the San Luis Obispo tower.

Response Spectra Development

Evaluation response spectra were developed based on both the 1991 NEHRP (FEMA 222) guidance and the current 1997 NEHRP (FEMA 302). FEMA 222 is used because the results of these evaluations should be consistent with the earlier Standard ATCT evaluation in UBC Seismic Zones 3 and 4. FEMA 302 is used because it is current guidance with an improved more refined definition of seismic hazard. These two spectra are compared and are found to produce seismic hazard definitions that are reasonably consistent with one another. Finally, the FEMA 302 spectrum definition of seismic hazard is used for all evaluations.

1991 NEHRP (FEMA 222) Based Response Spectra

The modal seismic design coefficient, C_{sm} is given by Equation 1 (from FEMA 222, Equation 5-3):

$$C_{sm} = \frac{1.2A_v S}{RT_m^{2/3}} \quad [\text{Eq 1}]$$

where:

A_v = the effective peak velocity-related acceleration in g's (from FEMA 222, Map 2), and a value of 0.4 g was used for each tower.

S = the site coefficient (from FEMA 222, Table 3.1). An S2 soil profile was used in earlier evaluations with a value of 1.2, and is again used here.

R = the response modification factor (defined in FEMA 222, Table 3.3). Because this evaluation is based on a pushover analysis procedure, it was decided to evaluate them assuming elastic response by assuming an R factor of 1.0.

T_m = the modal period of vibration in seconds for the m th mode of the building.

The modal seismic design coefficient, C_{sm} , is limited to a maximum value, C_{smMax} , given by Equation 2:

$$C_{smMax} = \frac{2.5A_a}{R} \quad [\text{Eq 2}]$$

where:

A_a = the effective peak acceleration in g's (from FEMA 222, Maps 1); a value of 0.4 g was used for each tower.

The FEMA 222-based evaluation spectra are plots of these modal seismic design coefficients, C_{sm} , shown in Figure 9.

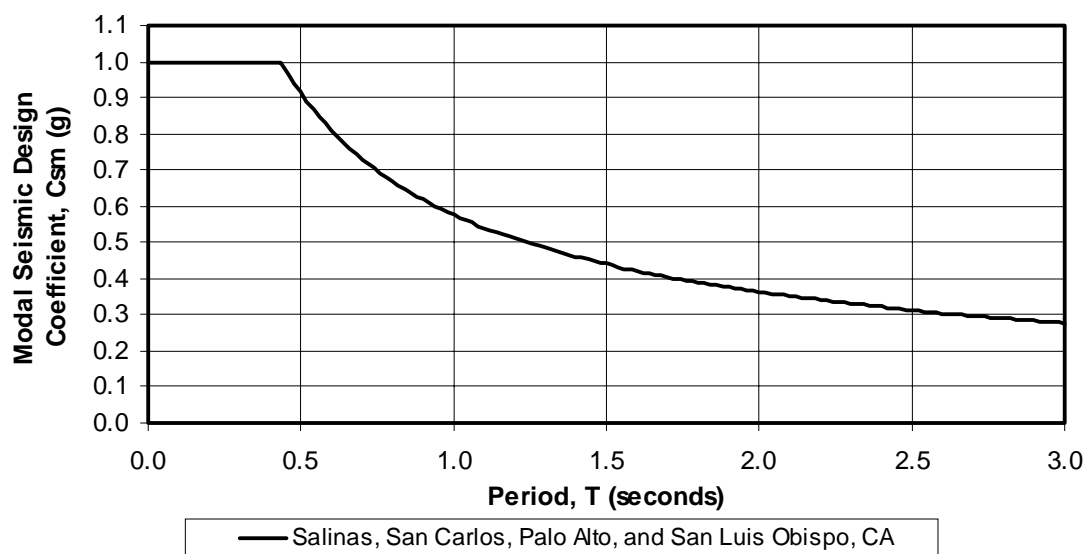


Figure 9. 1991 NEHRP-based evaluation response spectra, C_{sm}

1997 NEHRP (FEMA 302)-Based Response Spectra

The Seismic Performance Objective for all of these towers is Substantial Life Safety,* because all of them are noncritical power facilities. This performance objective equates to the Seismic Use Group I in FEMA 302. Because soil conditions are unknown, a Site Class D is conservatively assumed (FEMA 302, 4.1.2.1). The maximum considered earthquake design spectrum is defined for each tower based on the variables given in Table 5. These are based on the FEMA 302 design maps for short period, S_s , and at 1 second, S_1 , for 5 percent damped spectral response acceleration (FEMA 302, 4.1.1). The spectral quantities used to define these spectra are given in Table 5 for each tower. For Site Class D values of site coefficients, F_a and F_v are 1.0 and 1.5 respectively for all the towers being evaluated here (FEMA 302, Tables 4.1.2.4a and 4.1.2.4b).

* U.S. Army Corps of Engineers, Seismic Design for Buildings, TI 809-04, 1998.

Table 5. FAA response spectra calculations based on FEMA 302.

California Location	S_S (g)	S_1 (g)	S_{MS} (g)	S_{M1} (g)	S_{DS} (g)	S_{D1} (g)	T_0 (sec)	T_s (sec)
Salinas	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60
San Carlos	1.60	0.90	1.60	1.35	1.07	0.90	0.17	0.84
Palo Alto	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60
San Luis Obispo	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60

The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1 second (S_{M1}), adjusted for site class effects are calculated as follows (FEMA 302, Equations 4.1.2.4-1 and 4.1.2.4-2):

$$S_{MS} = F_a S_S \quad [\text{Eq 3}]$$

and

$$S_{M1} = F_v S_1 \quad [\text{Eq 4}]$$

These values define the elastic spectra. The values are reduced to define design earthquake spectral response acceleration at short periods, S_{DS} , and at 1-second period, S_{D1} as follows (FEMA 302, Equations 4.1.2.5-1 and 4.1.2.5-2):

$$S_{DS} = \frac{2}{3} S_{MS} \quad [\text{Eq 5}]$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \quad [\text{Eq 6}]$$

From these terms, design response spectra are developed for each of the tower locations. For the natural period of the structure (T), this spectrum defines values of effective acceleration. The three regions of this spectrum are defined as follows:

- For periods less than or equal to T_0 , the design spectral acceleration, S_a , shall be (FEMA 302, Equation 4.1.2.6-1):

$$S_a = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS} \quad [\text{Eq 7}]$$

- For periods greater than or equal to T_0 and less than or equal to T_s , the design spectral response acceleration, S_a , shall be taken as equal to S_{DS} .
- For periods greater than T_s , the design spectral response acceleration, S_a , shall be (FEMA 302, Equation 4.1.2.6-3):

$$S_a = \frac{S_{D1}}{T} \quad [\text{Eq 8}]$$

where

T = the fundamental period of the structure in seconds

$$T_0 = 0.2S_{D1}/S_{DS}$$

$$T_s = S_{D1}/S_{DS}$$

Figure 10 shows these design response spectra for the towers in Table 1. Reliability factors, ρ , were calculated, and found to be less than 1.0 for each tower, so that a value of 1.0 is used (FEMA 302, 5.2.4.2). This is a factor for the extent of structural redundancy.

The modal seismic response spectra, C_{sm} , are calculated from the design response spectra (Figure 10), using the following equation (FEMA 302, Equation 5.4.5-3):

$$C_{sm} = \frac{S_a}{R/I} \quad [\text{Eq 9}]$$

In these evaluations, R and I values of 1.0 are used so that the modal seismic response coefficient, C_{sm} , is the same as the design response spectra, S_a . However, R values of 1 are used for each tower because the response spectra generated is used for pushover analysis where the analysis assumes a linear response. Member and connections being evaluated may have demand-capacity ratios, DCRs as great as 2.0, and be considered to pass evaluation without further evaluation. For those members with DCRs greater than 2.0, hinges will be placed at joints, and further load applied.

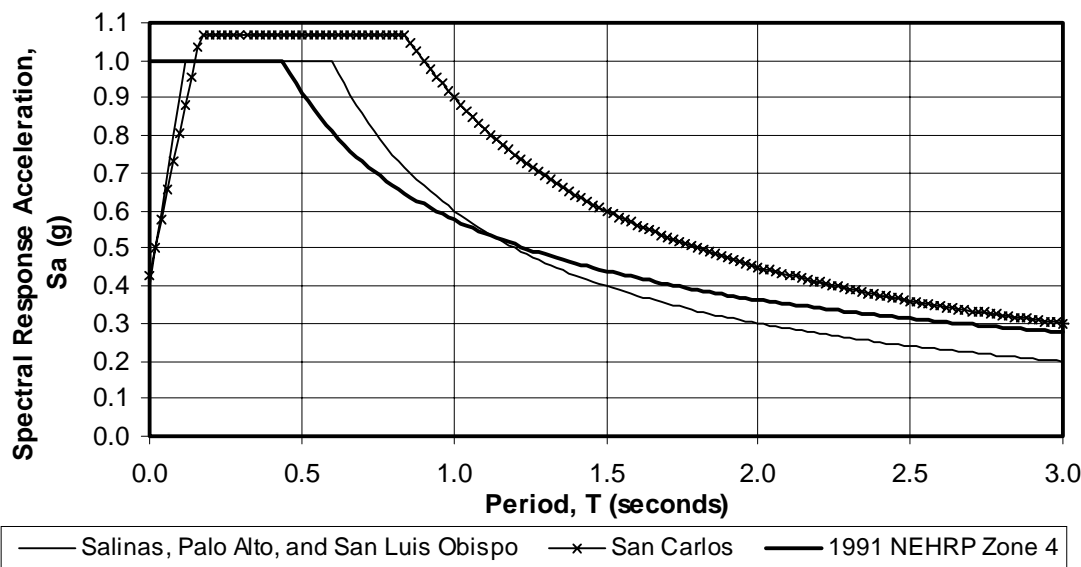


Figure 10. Maximum considered earthquake design response spectra.

Figure 10 shows the FEMA 222 spectrum plotted along with the FEMA 302 spectra for all the towers. This figure demonstrates that the new spectra are reasonably consistent with the spectrum used in the earlier evaluations. The San Carlos spectrum is an exception, where the spectral acceleration values are significantly greater at periods greater than 0.5 seconds. The primary modes that dominate the response of the cab are at periods of 1.27 and 0.78 seconds. At these periods, the spectral accelerations are 0.71 and 1.07 g for the FEMA 302 San Carlos spectrum, 0.47 and 0.77 g for the other FEMA 302 location spectra, and 0.49 and 0.68 g for the FEMA 222 spectrum. Modes of vibration that control response of the shaft are only slightly influenced by the difference in spectra because their periods (0.60 and 0.55 seconds for Salinas and 0.39 and 0.38 seconds for San Luis Obispo) fall within the period range where the spectral accelerations are 1.0 g (slightly less for FEMA 222). The San Carlos tower was evaluated with the spectrum shown in Figure 10, with a slightly greater spectral amplitude of 1.07 g for the periods that control the response of the shaft (0.45 and 0.28 seconds). However, for the Type L towers, the taller Salinas tower has a more critical shaft, so the higher amplitude San Carlos spectrum does not control the vulnerability of the Type L shaft components. Therefore, the vulnerability of the shaft components based on the FEMA 302 spectra is consistent with their vulnerability had they been based on FEMA 222 spectrum. In a similar manner, the difference in vulnerability of particular structural components, between the 1991 and 1997 NEHRP spectra, can be evaluated. For any member or connection, the mode(s) of vibration that dominate their vulnerability can be determined (based on mode shapes and participating mass), and the spectral acceleration between

the 1991 and 1997 spectra compared to assess that component's difference in vulnerability.

Load Combinations

Combination of Load effects were evaluated for the worst-case combinations of horizontal and vertical loads, together with gravity loads. The effect of seismic loads, E , when the horizontal and gravity loads are additive is represented by (FEMA 302, Equation 5.2.7-1):

$$E = \rho_x Q_E + 0.2S_{DS}D \quad [\text{Eq 10}]$$

where

E = the effect of horizontal and vertical earthquake-induced forces

D = the effect of dead load

Q_E = the effect of horizontal seismic forces.

For all the control towers, at all floor levels $\rho_x = 1$, so that

$$E = Q_E + 0.2S_{DS}D \quad [\text{Eq 11}]$$

The effect of seismic loads, E , when the horizontal and gravity loads counteract each other was represented by (FEMA 302, Equation 5.2.7-2):

$$E = \rho_x Q_E - 0.2S_{DS}D = Q_E - 0.2S_{DS}D \quad [\text{Eq 12}]$$

The gravity loads (Q_G) were combined as follows when the effects of gravity are additive with seismic loading (FEMA 273, 3.2.8, Equation 3.2):

$$Q_G = 1.1(Q_D + Q_L + Q_S) \quad [\text{Eq 13}]$$

When the effects of gravity counteract seismic loads, Q_G becomes (FEMA 273, Equation 3.3):

$$Q_G = 0.9Q_D \quad [\text{Eq 14}]$$

where:

Q_D = the full weight of the structure

Q_L = the effective live load equal to 25 percent of the unreduced design load plus 100 percent of partition walls and attached equipment

Q_S = the effective snow load, which is taken as 20 percent (25 percent in 1997 UBC, Section 1612.3.2) of the design snow load when the design snow load is at least 30 psf. For all tower locations shown in Table 1, the design snow load is less than 30 psf, so the effective snow load was zero for all towers being evaluated here.

The effects of gravity load (dead, live, and snow) and seismic forces were combined as follows when the effect of gravity and seismic loads are additive by combining Equations 11 and 13:

$$Q_G + E = 1.1(Q_D + Q_L + Q_S) + Q_E + 0.2S_{DS}D \quad [\text{Eq 15}]$$

The weight calculations summarized in Tables 2–4 include the actual dead load, Q_D , and live load, Q_L . These combined loads, Gravity, are the gravity loads, D , that were multiplied by $0.2S_{DS}$ to calculate the vertical seismic loads. The effective snow load, Q_S , is zero for all the tower locations in Table 1. Using the values of S_{DS} in Table 5, Equation 15 becomes the following for all the towers in Table 1, except for San Carlos:

$$Q_G + E = 1.1(\text{Gravity}) + Q_E + 0.2S_{DS}(\text{Gravity}) = 1.3(\text{Gravity}) + Q_E \quad [\text{Eq 16}]$$

The San Carlos tower has a spectrum response acceleration at short periods, S_{DS} of 1.07 g, so that Equation 15 becomes the following:

$$Q_G + E = 1.1(\text{Gravity}) + Q_E + 0.2S_{DS}(\text{Gravity}) = 1.314(\text{Gravity}) + Q_E \quad [\text{Eq 17}]$$

The effects of gravity load (dead, live, and snow) and seismic forces were combined as follows when the effects of gravity counteract seismic loads, by combining Equations 12 and 14:

$$Q_G - E = 0.9(Q_D + Q_L) - Q_E - 0.2S_{DS}D \quad [\text{Eq 18}]$$

The weight calculations summarized in Tables 2 through 4 include the actual dead load, Q_D , and live load, Q_L . Equation 18 includes the actual live loads, so that the combined loads were the same Gravity value as in Equations 16 and 17. Vertical seismic load is the same as in Equation 16, but with the opposite sign because seismic forces counteract the effects of gravity. Equation 18 became the following for all the towers in Table 1, except for San Carlos:

$$Q_G - E = 0.9(\text{Gravity}) - Q_E - 0.2S_{DS}(\text{Gravity}) = 0.7(\text{Gravity}) - Q_E \quad [\text{Eq 19}]$$

The San Carlos tower has spectral response acceleration at short periods, S_{DS} of 1.07 g, so that Equation 18 became the following:

$$Q_G - E = 0.9(\text{Gravity}) - Q_E - 0.2S_{DS}(\text{Gravity}) = 0.686(\text{Gravity}) - Q_E \quad [\text{Eq 20}]$$

The member and connection forces and deflections were calculated by combining the square root of the sum of the squares (SRSS) of the various modal contributions of the response spectrum analysis of each tower (FEMA 302, 5.4.8). The finite element software used for this analysis, SAP2000, has an option to combine the modal contributions based on the SRSS, and this feature was used here.

A response spectrum analysis was performed with a model that includes enough modes in both orthogonal (horizontal) directions so that at least 90 percent of the building mass was included in the participating mass (FEMA 302, 5.4.3). The modal contributions to calculating the member and connection forces, base reactions, and displacements were combined by the SRSS. Each tower was evaluated by applying 100 percent of the lateral response spectrum based loading in one direction and 30 percent of the spectrum in the perpendicular lateral direction (FEMA 273 3.2.7).

Story Drift and P-Delta Effects

Story drifts were calculated directly from this analysis. Story drifts are the maximum difference in displacements at one floor level, δ_x , and the floor level below, δ_{x-1} , for a given column. For the tower cab, this became the maximum difference

in displacement at the cab roof and floor for a given window corner mullion. These displacements are the SRSS of the calculated displacements in the x and y direction (orthogonal horizontal directions) in the model.* The x and y displacements were calculated by SAP2000 based on the SRSS of each of the modal contributions of displacements at the particular joint in question. In this elastic analysis (no pushover), the elastic deflections should be amplified as follows (FEMA 302, Equation 5.3.7.1):

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad [\text{Eq 21}]$$

where:

C_d = the deflection amplification factor—a value of 4.0 could be used for all towers (FEMA 302, Table 5.2.2)

δ_{xe} = the deflection determined by elastic analysis

I = the occupancy importance factor, which for Seismic Use Group I is 1.0.

The amplification expressed in Equation 21 applies only to the cab steel frame elements, as the concrete bent elements in the Type L towers remain essentially elastic, even based on an analysis with an R value of 1. However, had the analysis and resulting deflections been based on seismic forces defined by the R values given in FEMA 302, Table 5.2.2 (R value of at least 4), these deflections would be proportionately smaller. Therefore, the deflections used to evaluate drift limitations were modified as follows, where the effects of R value and deflection amplification factor, C_d , canceled each other out, because calculated elastic deflections were based on R values of 1:

$$\delta_x = \frac{C_d \delta_{xe}}{RI} = \frac{(4)\delta_{xe}}{(4)(1)} \quad [\text{Eq 22}]$$

* Table 11 gives examples of such displacements for the Salinas tower.

These deflections, for a given column, were used to calculate story drifts, Δ_c , as follows:

$$\Delta_c = \delta_x - \delta_{x-1} \quad [\text{Eq 23}]$$

These calculated story drifts must be less than the allowable story drifts, Δ_a , calculated as follows (FEMA 302, 5.2.8, Table 5.2.8 and 5.3.7.1):

$$\Delta_a = 0.020 h_{sx} \quad [\text{Eq 24}]$$

where:

h_{sx} = the story height below level x.

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ , as determined by Equation 25, is equal to or less than 0.10 (FEMA 302, 5.3.7.2):

$$\theta = \frac{P_x \Delta_c}{V_x h_{sx} C_d} \quad [\text{Eq 25}]$$

where:

P_x = the total weight above or at level x

V_x = the seismic shear force acting between level x and x – 1

C_d = the deflection amplification factor. A value of 1 is used because the story drifts, Δ_c , are based on deflections, δ_x , that have not been amplified with values of C_d as explained above.

For the towers evaluated here, the stability coefficient, θ , was well below 0.10 for the tower shafts and cabs. No correction to calculated story drifts, Δ_c , was needed for P-delta effects.

3 Analysis Steps and Cases

The following steps were taken to conduct an analysis of each controlling tower configuration or case using SAP2000 finite element software. Several cases were evaluated because different tower configurations or load directions were more critical for various members or connections than others, and this approach allowed a thorough evaluation of the Type L and San Luis Obispo tower vulnerabilities. Steps 1 through 3 were followed for each analysis case. The remaining steps would have been used for only the controlling cases if a push-over analysis were needed to evaluate the vulnerability of a tower. Such evaluations would be based on the analysis cases where a member or connection DCR both exceeds 2.0 and is the greatest for all cases of the tower being evaluated. Push-over analysis was not needed for any of the tower evaluations presented here, but the procedure is described because it shows the context for the analysis conducted. In the San Luis Obispo tower evaluation, many compression braces buckled. Step 5a was used in analysis case SLO3a to evaluate this tower by removing compression braces, so that the remaining braces were tension only.

Analysis Steps

The following steps outline the approach used to evaluate each controlling configuration of the Type L or San Luis Obispo towers. More than one step was taken in most SAP2000 analysis runs, and these steps are shown in terms of the Run number.

Run 1

1. Gravity load alone was first applied to the towers.
2. A modal analysis was conducted to define the periods and mode shapes. SAP2000 uses this information to define the distribution of lateral seismic loads applied to the structure. These loads are based on mode shapes and period of each mode and the effective acceleration based on the spectral acceleration for each mode defined by the response spectrum.

3. The model was analyzed for the horizontal earthquake ground motions using the response spectra analysis procedure. Vertical ground motions were accounted for by multiplying the gravity load effect by the factors defined in Equations 16, 17, 19, and 20.
4. When a member or connection has a DCR greater than 2.0, for the gravity (step 1) and full elastic earthquake spectra (step 3), a seismic factor (F_1) was calculated. This is the factor of seismic load that, together with gravity, gives a DCR equal to 1.0 for the failed component. The loads and deflections for the entire structure were recorded at this factored load. If, for example, all the corner mullions near their bases have a similar DCR and it is over 2.0, an F_1 value should be determined for which the average corner mullion DCR is equal to 1.0. This is the point at which mullion yielding occurred and a hinge has formed at the base of each mullion. This was done for the San Carlos Type L tower evaluation, where F_1 equaled 0.1055. However, this resulted in a collapse mechanism and push-over analysis was not needed.

Run 2

5. If needed the model would be modified to reflect the yielded condition and the analysis continued as summarized below:
 - a. If compression members buckle, their capacity in compression would be reduced to zero. Under this condition, these members would be removed. Modal analysis would be repeated with the modified model. The mode shapes and frequencies would be checked against the initial modal analysis (Run 1), to validate behavior. The new structure would be reanalyzed for the total gravity and seismic loads as presented below. However, for the control towers with relatively little redundancy, it is unlikely that they would be able to pick up the full gravity and seismic load after a critical highly stressed member buckles.
 - b. If members yield in tension or bending, the applied loads at these locations would not be allowed to increase further with increasing displacements (i.e., a flat stress versus strain curve for additional loading). This would be the case if, for example, the mullions fail in bending near their base. Hinges would be placed at these locations, and modal analysis would be repeated with the modified model. The mode shapes and periods would be checked against the initial modal analysis (Run 1), to validate behavior. The new structure would be analyzed for the additional seismic loads only (loads beyond the factored seismic loads, F_1). Member

loads from this analysis would be added to the factored load determined in Step 4, and new DCR values determined.

6. If needed, the seismic analysis would be repeated with the seismic loads only (both horizontal and vertical). If new yielding of members or connection failure occurs before the full seismic load is carried, a second seismic factor (F_2) is calculated for that point at which the second yielding occurs. The applied load would be factored up (F_3, \dots, F_n) until further yielding, collapse of the structure, or the full seismic loading is carried.
7. Steps 5 and 6 are repeated, as needed, until further yielding, collapse, or the full seismic load is carried.
8. The results of the various load steps were summed in a spreadsheet. The resulting applied loads for critical members and connections were compared against the capacity of that component in a MathCAD* model that represents the capacity of the component at its loaded condition.
9. Total lateral displacements were presented for the maximum displacement at each critical floor level or elevation in the tower.
10. If a push-over analysis was carried out, story shear would be plotted with respect to story drift (maximum horizontal displacement) at critical floors or elevations in the tower. Calculated drifts would be compared with allowable story drift for these structures ($0.02 h_{sx}$).

Analysis Cases

The model configurations and loading directions, θ_L (shown in Tables 6 and 7) were used to evaluate the Salinas and San Carlos Type L and San Luis Obispo braced frame towers.

* MathCAD software is a product of MathSoft Inc, Cambridge, MA.

Table 6. Type L Analysis Cases.

Case #	Critical Component	Tower Location	Tower Height	Windows Acting as Shear Walls	Full Spectrum Direction, θ_L^*
L1-50	Shaft	Salinas	50 ft	All	90 deg.
L2-50	Shaft	Salinas	50 ft	All	45 deg.
L3-50	Shaft	Salinas	50 ft	All	0 deg.
L4-50	Shaft	Salinas	50 ft	One	19.3 deg.
L5-30	Cab	San Carlos	30 ft	None	0 deg.
L6-30	Cab	San Carlos	30 ft	None	90 deg.
L7-30	Cab & Shaft	San Carlos	30 ft	One	19.3 deg.
L8-30	Cab & Shaft	San Carlos	30 ft	All	90 deg.
L9-30	Cab & Shaft	San Carlos	30 ft	All	45 deg.
L10-30	Cab & Shaft	San Carlos	30 ft	All	0 deg.
L11-50	Cab & Shaft	Salinas	50 ft	None	90 deg.
L12-30	Cab/upgrade	San Carlos	30 ft	None	90 deg.
L13-50	Shaft/upgrade	Salinas	50 ft	All	90 deg.

Table 7. San Luis Obispo Analysis Cases.

Case #	Critical Component	Tower Height	Windows Acting as Shear Walls	Full Spectrum Direction, θ_L^*
SLO1	Shaft	45 ft	All	90 deg.
SLO2	Shaft	45 ft	All	45 deg.
SLO3	Shaft	45 ft	All	0 deg.
SLO3a	Tension braces only	45 ft	All	0 deg.
SLO4	Cab & Shaft	45 ft	One	16.7 deg.
SLO5	Cab	45 ft	None	90 deg.
SLO6	Cab	45 ft	None	45 deg.
SLO7	Cab	45 ft	None	0 deg.

* Angle of primary loading, counter clockwise to the X-axis.

4 Type L Analysis Results

All Type L towers were analyzed using SAP2000 and MathCAD member and connection evaluation files. The SAP2000 analysis modeled the towers and defined component demand in terms of displacements and member forces and moments. The MathCAD files determine member and connection capacity and demand capacity ratios, DCR, based on the SAP2000 forces and moments. The first section of these results focuses on the tower shaft evaluation, which is controlled by the taller 50-ft Salinas tower. The second section focuses on the cab evaluation, which is controlled by the shorter 30-ft San Carlos tower. Table 6 summarized all the cases used to evaluate the Type L towers. Each section in this report presents the modal analysis results, deflections, story drifts, forces, and moments of critical members and connections, and the resulting DCR. The towers may fail evaluation based on either exceeding drift limits, or high DCRs that lead to collapse of the towers. Code-based design resistance factors have been dropped (i.e., set to 1.0) for the purpose of evaluating structural members. The design resistance factors are included in the evaluation of connections, as such failure must be prevented because they would fail in a more brittle manner than the structural members.

Salinas Tower – Shaft Members and Connections

The most critical condition in the Salinas shaft evaluation is the extreme condition where the cab windows all remain intact and act as fully effective shear walls (analysis Cases L1 through L3 and L13). This will decrease the fundamental period of the structure and will increase the effective acceleration and therefore load the shaft members and connections more severely. The assumption of the windows acting as fully effective shear walls is unrealistic, but it provides an upper bound basis for evaluating the effect of the windows remaining intact. This effect is modeled by defining the shear stiffness of the window if it acts as a shear block. The properties of all members around the window perimeters are then increased so their bending stiffness equals the combined stiffness of the windows as shear blocks, plus the actual bending stiffness of the members. In the actual Type L tower cabs, the connections between the mullions and channels above and below the windows are pinned, but these are changed to “fixed” to represent the effect of the windows acting as shear walls. Analysis Cases L1, L2,

and L3 evaluate the performance of the Salinas tower with the windows acting as shear walls with the full seismic response spectrum acting at 90, 45, and 0 degrees to the X-axis (θ_i). The X-axis orientation is shown on the model plot in Figure 4. This axis is along the front face of the tower as shown on Drawing S2 (Corlett & Spackman 1966) for all three Type L towers.

Another possible case is the condition where only one window remains intact. Case L4 represents this condition where only one window acts as a shear wall. The location of the center of mass (for 5 percent accidental eccentricity) is placed farthest away from the plane of this shear wall window. This will create the greatest distance between the center of mass and center of stiffness in the plan of the building and will create the greatest torsional response of the tower shaft. The resultant direction of the 100 percent full seismic spectrum loading plus the 30 percent orthogonal spectrum is parallel to this shear wall window, to create the worst torsional response. The window mullion at the stairway (Col 5 on Drawing S3 and S4, section 9; Corlett & Spackman 1966) has a much deeper cross-section than the other mullions. This mullion is 12-in. deep at the base compared to the others, which are 7 in. Therefore, this mullion tends to attract more load. This shear wall mullion is placed along side the stiffer mullion to further increase the distance between the center of stiffness and center of mass in plan.

Finally, Case L11 is the 50-ft Salinas tower without any shear walls. This model was evaluated for the full seismic spectrum at only 90 degrees because this provided the most severe loading for critical shaft components.

Modal Analysis Results and Deflections

Table 8 presents the primary modes of vibration for the Type L, Cases L1, L2, and L3 evaluations. The cumulative participating mass shows that a much greater portion of the mass participates in the first X and Y lateral modes than in the L11 case (Table 9), for the same tower without the windows acting as shear walls. This demonstrates that the cab is more coupled with the tower shaft vibration in the first mode, and this will more heavily load the shaft components due to higher effective accelerations.

Table 10 summarizes maximum lateral deflections at each floor level and other key locations in the control towers for the shaft evaluation cases. All deflections are the SRSS of the total X and Y lateral deflections. The shaft deflections are greater for the shear wall window cases (L1 – L3) because of the coupling and higher effective accelerations described above. Cab deflections for the cases of

Table 8. Salinas (Type L – 50 ft) Shaft Evaluation Modal Analysis Results, L1, L2, and L3.

Mode #	L1, L2, L3				Mode of Vibration
	Period (sec)	Cumulative Participating Mass (%)			
		X-dir	Y-dir	Z-dir	
1	0.724	5.2	0.7	0.0	1 st Torsion
2	0.603	5.5	52.0	0.1	1 st Y-Lateral
3	0.549	59.7	52.0	0.1	1 st X-Lateral
4	0.353	63.4	59.1	0.3	2 nd Torsion/Y-Lateral
5	0.341	65.7	67.4	0.8	2 nd Y-Lateral/Torsion
6	0.211	67.9	67.4	0.8	2 nd X-Lateral/Torsion
7	0.176	67.9	68.6	11.0	1 st Vertical/cab floor & roof
8	0.166	68.1	68.6	11.1	3 rd Torsion
9	0.134	68.8	69.1	11.1	2 nd Vertical/cab roof & floor
10	0.124	74.7	69.5	11.3	3 rd X-Lateral/cab rocking
11	0.119	74.7	75.1	12.0	3 rd Y-Lateral/cab rocking
12	0.108	75.9	80.3	12.0	4 th Y-Lateral/cab vertical
13	0.105	80.1	80.4	12.1	4 th X-Lateral/cab vertical
17	0.097	83.6	81.6	20.3	3 rd Vertical/4 th floor & roof
20	0.094	83.6	82.5	31.3	4 th Vertical/2 nd & 3 rd floor
31	0.077	85.2	85.6	39.6	4 th Torsion
56	0.048	87.5	92.3	46.6	4 th Y-Lateral
58	0.046	94.0	93.7	46.6	4 th X-Lateral

Table 9. Salinas (Type L – 50 ft) Shaft Evaluation Modal Analysis Results, L4 and L11.

Mode #	L11				Mode of Vibration	L4 Period (sec)
	Period (sec)	Cumulative Participating Mass (%)				
		X-dir	Y-dir	Z-dir		
1	1.351	4.6	3.5	0.0	1 st Torsion	1.342
2	1.300	6.6	12.6	0.0	1 st Y-Lateral	1.006
3	0.828	14.8	12.6	0.0	1 st X-Lateral	
4	0.550	18.9	12.9	0.0	2 nd Torsion/X-Lateral	0.653
5	0.468	21.5	64.2	0.1	2 nd Y-Lateral	0.500
6	0.463	66.0	66.7	0.1	2 nd X-Lateral	0.463
11	0.206	66.7	66.8	10.0	1 st Vertical/cab floor & roof	0.204
12	0.189	67.3	66.9	10.0	3 rd Torsion	
25	0.113	74.6	71.8	12.9	3 rd X-Lateral/cab rocking	0.097
26	0.111	81.5	74.6	13.0	3 rd X-Lateral/cab rocking	0.112
27	0.106	81.8	79.7	13.1	3 rd Y-Lateral/cab rocking	0.108
32	0.097	83.2	81.2	21.5	2 nd Vertical/4 th floor	0.094
34	0.095	83.2	85.3	21.9	3 rd Y-Lateral/cab vertical	
35	0.094	83.2	85.3	32.4	3 rd Vertical/2 nd & 3 rd floor	0.094
39	0.091	83.3	85.5	38.1	4 th Vertical/shaft roof	0.091
44	0.076	85.3	85.6	38.6	4 th Torsion	0.077
69	0.048	87.5	92.1	46.6	4 th Y-Lateral	0.048
71	0.046	94.0	93.7	46.6	4 th X-Lateral	0.046

Table 10. Salinas Shaft Evaluation Selected Horizontal SRSS Deflections.

Location	L1		L2		L3		L4		L11	
	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	204	0.25	204	0.31	204	0.35	212	0.34	204	0.21
3 rd Floor, δ_2	304	1.00	304	1.21	304	1.37	312	1.29	304	0.86
4 th Floor, δ_3	404	2.15	404	2.52	404	2.83	412	2.61	404	1.85
Shaft Roof, δ_4	508	3.49	508	4.01	508	4.46	517	4.01	508	2.96
Cab Floor, δ_5	607	4.98	603	4.62	601	4.63	610	4.02	607	3.74
Bottom of Windows	706	6.17	706	5.38	701	5.02	708	5.45	706	5.92
Top of Windows	810	7.16	810	6.14	801	5.44	818	10.67	814	11.72
Cab Roof, δ_6	911	7.50	911	6.38	901	5.59	921	12.14	916	13.33

shear wall windows are unrealistically low because of the shear wall window stiffening. These cases are intended for shaft evaluation only, and will produce unrealistic results in the cab. Case L11, without the shear wall windows, shows that the cab will experience significant deflections. These deflections are even greater for the San Carlos tower cab evaluation, which will be discussed in the section for that tower.

Table 11 presents the Salinas tower story drifts. The only level exceeding the acceptable limits is in the cab between the shaft roof and cab floor. Drifts are much greater in the San Carlos cab evaluation and will be discussed in that section of this report.

Table 11. Salinas Tower Story Drift and P-delta Effect Evaluation.

Location	Story	Story	Allow	Calc			Gravity	Seismic	Story		
	Elev	Height	Story	Analysis	Elastic	Story	Load	Shear	Stability	Drift w/	
	Y_x	$h_{sx} = Y_x - Y_{x-1}$	Drift	Case	Joint	Defl	Drift	above Y_x	Force	Coeff	P-delta
	Y_x	$Y_x - Y_{x-1}$	Δ_a		#	δ_x	Δ_c	P_x	V_x	θ	Δ_c
	(in.)	(in.)	(in.)			(in.)	(in.)	(kips)	(kips)		(in.)
Ground Floor	0			L3	102	0.00					
2nd Floor	134	134	2.68	L3	204	0.35	0.35				
3rd Floor	278	144	2.88	L3	304	1.37	1.02				
4th Floor	422	144	2.88	L3	404	2.83	1.46				
Shaft Roof	582	160	3.2	L3	508	4.46	1.63				
Shaft Roof	582	160	3.2	L1	513	3.43					
Cab Floor	620	38	0.76	L1	607	4.98	1.55	60	115	0.021	1.55
Cab Roof	777	157	3.14								

Shaft Member and Connection Evaluation

Tables 12 through 16 present the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. The force, moment, and DCR tables (e.g., Tables 12 through 16) contain the following information in table columns (left to right) as defined below:

- Component = the description of the critical building member or connection being evaluated.
- Drawing/Section # = the building drawing and section number that defines the component being evaluated. When multiple members are being evaluated, the critical member is underlined in the column.
- Member ID/End = the building finite element model member number and joint number for the most critically loaded (in terms of DCR) member or connection of the particular component being evaluated.
- Length = the length in inches when the component being evaluated is a building member.
- Load Type = the description of the type of load for which the forces and moments are determined – i.e., Gravity, horizontal seismic forces, and vertical seismic forces (either 30 or 31.4 percent of gravity).
- P = the axial force in kips where tension is positive.
- V2 or Vy = the shear force in kips applied to the component in the strong axis of the member from which the force was obtained.
- V3 or Vx = the shear force in kips applied to the component in the weak axis of the member from which the force was obtained.
- T = the torsional moment in kip-inches applied to the component from the end of member for which the torsion was obtained.
- M2 or My = the moment in kip-inches applied to the component in the weak axis of the member from which the force was obtained.
- M3 or Mx = the moment in kip-inches applied to the component in the strong axis of the member from which the force was obtained.

- DCR = the demand capacity ratio. The component being evaluated meets the requirements of this life-safety evaluation if the DCR values fall below 2.0.
- App # = the appendix number for the MathCAD files that define the capacity definition for the component being evaluated. The total applied forces and moments presented in these tables are entered into these files to define the DCR for these particular forces and moments. Appendices B1 through B14 define the capacity of all Type L tower components being evaluated using the forces and moments from the L1 analysis only. Tables 13 through 15; 20 through 25; 29 and 34 reference the same Appendices but use different forces and moments to determine their DCRs. Appendix B15 defines the capacity of a structural tube member used in the tower retrofit. Values from this evaluation are only used in the upgrade analysis as presented in Tables 29 and 34. The tables and the report Table of Contents define the particular component for which capacity is determined in each of the appendices.

Case L1 with the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90^\circ$) gives the worst loading for critical shaft members and connections. The highest shaft DCR is at the horizontal portion of the column bent, at the bent corner at the roof. This has a DCR of 2.11, which is above 2.0. As was described earlier, DCRs up to 2.0 are acceptable because of the conservative use of an R value of 1.0 in the analysis. The highest DCR of 2.11 is with the full seismic loading at 55.3 degrees and with one window acting as a shear wall (case L4). The conservative assumption of the shear walls acting as fully effective shear walls results in loading the horizontal portion of the bent much more heavily. This influence can be seen by comparing the maximum DCR of this member in the L1 and L11 analyses cases. The L1 and L11 analysis are identical except that the L1 assumes all windows act as shear walls and this yields a DCR of 2.03, whereas L11 assumes no shear walls and this yields a DCR of 1.69. The shear wall assumption is very conservative, and the horizontal portion of the bent, as well as all other members and connections of the shaft, are therefore judged to meet the requirements of this life-safety evaluation. Tables 12 through 16 show that corner mullions and the channels above and below windows have DCRs well above 2.0. These members are in the cab and cab elements will be evaluated in the San Carlos tower evaluation, which is even more critical for cab components.

Table 12. Forces, Moments, and DCRs (L1, 50-ft tower, all windows, $\theta_L = 90^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 12	134.1	Gravity	-111.3	1.4	-0.1	0	-5	211		
				H. Seismic	69.8	93.3	-10.6	862	-3470	37971		
				0.300 Gravity	33.4	0.4	0.0	0	-2	63		
(counteract)		n=102		Total	-8.1	95.1	-10.7	862	-3477	38245	0.96	B1
Bent Column at Shaft	S1	NPCM 42	160.1	Gravity	-26.9	8.2	0.0	-2	-15	-817		
				H. Seismic	69.9	72.4	3.2	-438	-357	-7848		
				0.300 Gravity	8.1	2.5	0.0	-1	-5	-245		
(counteract)		n=536		Total	51.1	83.0	3.2	-440	-377	-8910	1.15	B2
Bent Beam at Shaft	S1	TBNT 532	92.9	Gravity	-5.3	-7.4	0.2	1	0	-531		
				H. Seismic	56.2	-45.6	2.3	381	429	-7600		
				0.300 Gravity	1.6	-2.2	0.0	0	0	-159		
(counteract)		n=508		Total	52.5	-55.2	2.5	382	430	-8290	2.03	B3
Bent conn at center of tower roof	S1/2,3	TBNT 534	-	Gravity	-6.8	0.9	0.1	1	0	0		
				H. Seismic	-11.4	44.3	3.2	382	0	0		
(additive)		n=531		0.300 Gravity	-2.0	0.3	0.0	0	0	0		
				Total	-20.2	45.4	3.3	383	0	0	0.48	B4
Bent/roof beam conn	S2	EB	-	Gravity	-2.9	14.6	-0.3	2	0	0		
	S4/13,19	509		H. Seismic	-24.0	31.9	-4.8	5	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.9	4.4	-0.1	1	0	0		
(additive)		n=514		Total	-27.8	50.8	-5.2	8	0	0	0.59	B5
Corner mullions at their base	S4/A,2,3	CLCM 1	37.9	Gravity	-10.5	-0.8	2.4	0	-4	-34		
				H. Seismic	-23.3	-19.7	9.7	54	-242	-670		
				0.300 Gravity	-3.2	-0.2	0.7	0	-1	-10		
(additive)	col. 1-4	n=601		Total	-36.9	-20.6	12.8	55	-246	-714	1.13	B6
Corner mullion/ 10WF conn	S4/A,2,5	CLCM 4	-	Gravity	-13.7	3.0	0.2	-1	6	144		
				H. Seismic	14.1	20.4	10.1	-68	59	617		
				0.300 Gravity	4.1	0.9	0.0	0	2	43		
(counteract)	col. 1-4	n=534		Total	4.5	24.3	10.3	-70	66	804	6.15	B7
Corner mullion/ 10WF conn	S4/8,9	CLCM 3	-	Gravity	-8.2	0.8	-2.6	-2	-94	-67		
				H. Seismic	29.3	25.5	-11.6	-46	-288	-207		
				0.300 Gravity	2.5	0.2	-0.8	-1	-28	-20		
(counteract)	col. 5	n=506		Total	23.5	26.5	-15.0	-48	-410	-294	2.07	B8
10WF72 at 16WF88 shaft roof	S2,S3	RAD	43.4	Gravity	-1.8	16.2	0.1	0	-3	-551		
	S4/A	525		H. Seismic	-27.4	35.6	4.7	22	-230	-2385		
				0.300 Gravity	-0.5	4.9	0.0	0	-1	-165		
(additive)		n=515		Total	-29.7	56.6	4.8	22	-233	-3101	1.19	B9
Cab floor 12WF27 near Col 5	S2	CEPG 602	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
				H. Seismic	13.6	-2.3	1.0	0	-65	142		
				0.300 Gravity	0.8	-0.8	0.1	0	-3	57		
(additive)		n=604		Total	16.8	-5.8	1.3	0	-79	390	0.73	B10
C 4 x 7.25 at base of window	S3	CEPG 704	192.0	Gravity	-2.1	0.1	-0.1	0	0	6		
				H. Seismic	-16.6	2.4	-0.8	0	0	115		
				0.300 Gravity	-0.6	0.0	0.0	0	0	2		
(additive)		n=704		Total	-19.3	2.5	-0.9	0	0	123	2.98	B11
Interior mullion (S3x7.5)	A5/5	WM 2	90.4	Gravity	0.5	0.0	0	0	0	0		
				H. Seismic	-16.4	0.0	0	1	0	1		
				0.300 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	-16.0	0.0	0	1	0	1	0.90	B12
Corner mullion at the roof	S4/1,2,3,7	CUCM 7	26.7	Gravity	-2.1	0.2	-0.3	-5	-6	6		
				H. Seismic	-2.2	9.6	-5.0	-43	-142	257		
				0.300 Gravity	-0.6	0.1	-0.1	-2	-2	2		
(additive)		n=805		Total	-4.9	9.9	-5.3	-50	-150	266	0.50	B13
C 6 x 8.2 at top of window	S3	CEPG 818	237.0	Gravity	0.9	-0.7	-0.1	0	0	-27		
				H. Seismic	-13.1	-8.5	-5.1	1	0	-352		
				0.300 Gravity	-0.3	-0.2	0.0	0	0	-8		
(buckling)		n=818		Total	-12.5	-9.4	-5.2	1	0	-387	2.54	B14

Table 13. Forces, Moments, and DCRs (L2, 50-ft tower, all windows, $\theta_L = 45^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base (counteract)	S1	NPCM 12 n=102	134.1	Gravity H. Seismic 0.300 Gravity Total	-111.3 68.4 33.4 -9.5	1.4 87.6 0.4 89.5	-0.1 -16.3 0.0 -16.3	0 1454 0 1454	-5 -4972 -2 -4979	211 35774 63 36048	1.05	B1
Bent Column at Shaft (counteract)	S1	NPCM 42 n=536	160.1	Gravity H. Seismic 0.300 Gravity Total	-26.9 68.4 8.1 49.6	8.2 63.0 2.5 73.7	0.0 4.8 0.0 4.8	-2 -858 -1 -860	-15 -770 -5 -790	-817 -6956 -245 -8018	1.14	B2
Bent Beam at Shaft (counteract)	S1	TBNT 529 n=501	92.9	Gravity H. Seismic 0.300 Gravity Total	-4.7 35.6 1.4 32.3	-7.0 -38.5 -2.1 -47.6	0.0 -10.6 0.0 -10.6	-3 -742 -1 -746	-3 -1431 -1 -1435	-467 -6499 -140 -7106	2.00	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 538 n=531	-	Gravity H. Seismic 0.300 Gravity Total	-6.7 -6.1 -2.0 -14.8	-1.0 -41.2 -0.3 -42.4	0.0 -7.1 0.0 -7.2	1 727 0 727	0 0 0 0	0 0 0 0	0.44	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 509 n=514	-	Gravity H. Seismic 0.300 Gravity Total	-2.9 -17.4 -0.9 -21.2	14.6 22.7 4.4 41.6	-0.3 -3.3 -0.1 -3.7	2 4 1 6	0 0 0 0	0 0 0 0	0.48	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 3 n=603	37.9	Gravity H. Seismic 0.300 Gravity Total	-8.0 -28.9 -2.4 -39.3	0.8 19.5 0.2 20.6	-2.6 -12.4 -0.8 -15.7	-2 -70 -0.6 -72	4 222 1.2 227	-98 -687 -29.5 -814	1.19	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 5 n=532	-	Gravity H. Seismic 0.300 Gravity Total	-12.5 13.0 3.7 4.2	2.0 20.7 0.6 23.4	-0.2 -7.2 -0.1 -7.5	1 76 0 78	-6 -44 -2 -51	119 639 36 793	6.02	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3 n=506	-	Gravity H. Seismic 0.300 Gravity Total	-8.9 28.9 2.7 22.7	0.8 19.5 0.2 20.6	-2.6 -12.4 -0.8 -15.7	-2 -70 -1 -72	-94 -469 -28 -590	-67 -344 -20 -431	2.98	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 525 n=515	43.4	Gravity H. Seismic 0.300 Gravity Total	-1.8 -19.5 -0.5 -21.8	16.2 25.3 4.9 46.3	0.1 7.5 0.0 7.6	0 31 0 31	-3 -346 -1 -350	-551 -1692 -165 -2408	1.04	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602 n=604	63.0	Gravity H. Seismic 0.300 Gravity Total	2.5 10.4 0.8 13.6	-2.7 -1.4 -0.8 -5.0	0.2 0.8 0.1 1.0	0 0 0 1	-11 -51 -3 -64	191 91 57 339	0.61	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 712 n=712	192.0	Gravity H. Seismic 0.300 Gravity Total	-1.1 -20.2 -0.3 -21.6	0.1 1.8 0.0 1.9	-0.1 -12.9 0.0 -13.1	0 0 0 0	0 0 0 0	2 90 1 93	2.77	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2 n=807	90.4	Gravity H. Seismic 0.300 Gravity Total	0.5 -11.4 -0.1 -11.1	0.0 0.0 0.0 0.0	0 0 0 0	0 1 0 1	0 0 0 0	0 1 0 0	0.63	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 6 n=801	26.7	Gravity H. Seismic 0.300 Gravity Total	-2.5 -2.5 -0.7 -5.7	0.1 11.0 0.0 11.1	0.3 5.5 0.1 6.0	4 37 1 43	8 156 2 166	2 293 1 296	0.56	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 808 n=808	237.0	Gravity H. Seismic 0.300 Gravity Total	0.8 -10.2 -0.2 -9.7	0.6 6.3 0.2 7.0	0.1 3.8 0.0 4.0	0 0 0 0	0 0 0 0	21 248 6 275	1.74	B14

Table 14. Forces, Moments, and DCRs (L3, 50-ft tower, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 11	134.1	Gravity	-119.1	1.3	0.1	-2	20	-1		
				H. Seismic	73.52	92.2	19.9	-1656	5782	-37365		
				0.300 Gravity	35.7	0.4	0.0	0	6	0		
(counteract)		n=101		Total	-9.9	93.8	20.1	-1658	5808	-37366	1.15	B1
Bent Column at Shaft	S1	NPCM 41	160.1	Gravity	-30.0	7.6	-0.1	3	19	-800		
				H. Seismic	73.6	61.2	-5.8	1704	938	-6856		
				0.300 Gravity	9.0	2.3	0.0	1	6	-240		
(counteract)		n=535		Total	52.6	71.0	-5.9	1709	963	-7896	1.17	B2
Bent Beam at Shaft	S1	TBNT 529	92.9	Gravity	-4.7	-7.0	0.0	-3	-3	-467		
				H. Seismic	26.8	-37.7	11.9	-836	-1632	-6466		
				0.300 Gravity	1.4	-2.1	0.0	-1	-1	-140		
(counteract)		n=501		Total	23.5	-46.8	11.9	-840	-1635	-7073	2.03	B3
Bent conn at center of tower roof	S1/2,3	TBNT 536	-	Gravity	-5.1	-4.0	-0.1	-3	-4	-144		
				H. Seismic	-6.4	-42.2	-7.6	-834	0	0		
				0.300 Gravity	-1.5	-1.2	0.0	-1	-1	-43		
(additive)		n=531		Total	-13.0	-47.4	-7.7	-838	-5	-188	0.50	B4
Bent/roof beam conn	S2	EB 510	-	Gravity	-2.9	-14.9	0.3	-2	0	0		
	S4/13,19			H. Seismic	-9.5	-12.5	2.1	-3	0	0		
	S5/17,18			0.300 Gravity	-0.9	-4.5	0.1	-1	0	0		
(additive)		n=516		Total	-13.2	-31.9	2.4	-5	0	0	0.37	B5
Corner mullions at their base	S4/A,2,3	CLCM 3	37.9	Gravity	-8.0	0.8	-2.6	-2	4	-98		
				H. Seismic	-31.2	14.9	-12.4	-81	146	-402		
				0.300 Gravity	-2.4	0.2	-0.8	-0.6	1.2	-29.5		
(additive)	col. 1-4	n=603		Total	-41.5	16.0	-15.8	-84	151	-530	0.80	B6
Corner mullion/ 10WF conn	S4/A,2,5	CLCM 1	-	Gravity	-10.7	-0.8	2.4	0	88	-63		
				H. Seismic	32.9	-16.6	13.6	68	485	-359		
				0.300 Gravity	3.2	-0.2	0.7	0	26	-19		
(counteract)	col. 1-4	n=503		Total	25.5	-17.6	16.7	68	599	-441	6.51	B7
Corner mullion/ 10WF conn	S4/8,9	CLCM 3	-	Gravity	-8.2	0.8	-2.6	-2	-94	-67		
				H. Seismic	31.2	14.9	-12.4	-81	-542	-400		
				0.300 Gravity	2.5	0.2	-0.8	-1	-28	-20		
(counteract)	col. 5	n=506		Total	25.4	16.0	-15.8	-84	-664	-487	3.42	B8
10WF72 at 16WF88 shaft roof	S2,S3	RAD 525	43.4	Gravity	-1.8	16.2	0.1	0	-3	-551		
	S4/A			H. Seismic	-9.7	14.2	8.5	33	-385	-903		
				0.300 Gravity	-0.5	4.9	0.0	0	-1	-165		
(additive)		n=515		Total	-12.0	35.3	8.6	33	-388	-1619	0.81	B9
Cab floor 12WF27 near Col 5	S2	CEPG 602	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
				H. Seismic	6.9	-1.7	0.5	0	-33	108		
				0.300 Gravity	0.8	-0.8	0.1	0	-3	57		
(additive)		n=604		Total	10.2	-5.3	0.7	0	-46	356	0.49	B10
C 4 x 7.25 at base of window	S3	CEPG 712	192.0	Gravity	-1.1	0.1	-0.1	0	0	2		
				H. Seismic	-20.7	1.3	-7.7	-1	0	68		
				0.300 Gravity	-0.3	0.0	0.0	0	0	1		
(additive)		n=712		Total	-22.1	1.4	-7.9	-1	0	70	2.33	B11
Interior mullion (S3x7.5)	A5/5	WM 5	90.4	Gravity	-0.2	0.0	0	0	0	0		
				H. Seismic	-6.9	0.0	0	-1	0	0		
				0.300 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=712		Total	-7.1	0.0	0	-1	0	0	0.41	B12
Corner mullion at the roof	S4/1,2,3,7	CUCM 7	26.7	Gravity	-2.1	0.2	-0.3	-5	-6	-311		
				H. Seismic	-3.0	11.7	-5.2	-39	-146	-2		
				0.300 Gravity	-0.6	0.1	-0.1	-2	-2	-2		
(additive)		n=805		Total	-5.8	11.9	-5.5	-46	-154	-320	0.56	B13
C 6 x 8.2 at top of window	S3	CEPG 819	237.0	Gravity	0.9	-0.6	-0.1	0	0	25		
				H. Seismic	-7.7	-4.0	-2.5	0	0	159		
				0.300 Gravity	-0.3	-0.2	0.0	0	0	7		
(buckling)		n=820		Total	-7.0	-4.8	-2.6	0	0	191	1.23	B14

Table 15. Forces, Moments, and DCRs (L4, 50-ft tower, one window, $\theta_L = 55.3^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 11	134.1	Gravity	-119.3	1.3	0.1	-2	20	-3		
				H. Seismic	49.1	106.9	18.8	-1548	4503	-41335		
				0.300 Gravity	35.8	0.4	0.0	0	6	-1		
(counteract)		101		Total	-34.5	108.5	19.0	-1550	4529	-41338	1.08	B1
Bent Column at Shaft	S1	NPCM 41	160.1	Gravity	-30.2	7.3	-0.1	4	19	-770		
				H. Seismic	48.9	61.0	-5.2	1697	835	-7020		
				0.300 Gravity	9.1	2.2	0.0	1	6	-231		
(counteract)		n=535		Total	27.7	70.5	-5.2	1702	859	-8021	1.12	B2
Bent Beam at Shaft	S1	TBNT 529	92.9	Gravity	-4.3	-6.7	0.0	-3	-3	-432		
				H. Seismic	25.5	-41.0	11.3	-811	-1615	-6911		
				0.300 Gravity	1.3	-2.0	0.0	-1	-1	-130		
(counteract)		n=501		Total	22.5	-49.8	11.3	-815	-1619	-7472	2.11	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 536	-	Gravity	-4.5	-1.2	-0.1	-3	0	0		
				H. Seismic	-19.3	-41.0	-7.7	-810	0	0		
				0.300 Gravity	-1.3	-0.4	0.0	-1	0	0		
		n=531		Total	-25.1	-42.6	-7.8	-814	0	0	0.45	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 510	-	Gravity	-2.6	-14.7	0.1	-2	0	0		
				H. Seismic	-13.8	-7.6	2.5	-4	0	0		
				0.300 Gravity	-0.8	-4.4	0.0	-1	0	0		
		n=516		Total	-17.2	-26.6	2.6	-7	0	0	0.31	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 4	37.9	Gravity	-13.7	3.1	-0.3	-4	19	-9		
				H. Seismic	-4.0	17.2	-7.0	-128	294	-468		
				0.300 Gravity	-4.1	0.9	-0.1	-1.3	5.6	-2.6		
		n=610		Total	-21.8	21.3	-7.4	-133	318	-479	1.01	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 5	-	Gravity	-13.3	1.4	-0.4	-1	-7	115		
				H. Seismic	19.9	23.0	-3.5	-127	-41	1281		
				0.300 Gravity	4.0	0.4	-0.1	0	-2	34		
		n=532		Total	10.6	24.8	-4.0	-128	-50	1430	12.7	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3	-	Gravity	-8.2	0.4	-2.0	-15	-53	-36		
				H. Seismic	17.4	5.6	-17.7	-13	-806	-584		
				0.300 Gravity	2.5	0.1	-0.6	-4	-16	-11		
		n=506		Total	11.7	6.1	-20.2	-32	-874	-631	4.08	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 523	30.8	Gravity	-1.4	13.5	-0.4	-7	10	-299		
				H. Seismic	-23.0	19.9	-3.6	-41	99	-1365		
				0.300 Gravity	-0.4	4.1	-0.1	-2	3	-90		
		n=511		Total	-24.8	37.5	-4.0	-50	111	-1754	0.67	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602	63.0	Gravity	1.7	-2.7	0.2	0	-14	191		
				H. Seismic	11.8	-1.5	0.5	1	-31	95		
				0.300 Gravity	0.5	-0.8	0.1	0	-4	57		
		n=604		Total	14.0	-5.1	0.8	1	-49	343	0.50	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 703	192.0	Gravity	-1.0	0.0	-0.3	0	0	-5		
				H. Seismic	-7.1	-2.0	-12.4	0	0	-106		
				0.300 Gravity	-0.3	0.0	0.9	1	0	-1		
		n=703		Total	-8.3	-2.1	-11.8	1	0	-112	1.55	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2	90.4	Gravity	-0.6	0.0	0	0	0	0		
				H. Seismic	-11.0	0.0	0	-2	0	0		
				0.300 Gravity	-0.2	0.0	0	0	0	0		
		n=807		Total	-11.7	0.0	0	-2	0	0	0.65	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 9	26.7	Gravity	-5.8	0.3	-0.5	-9	-10	7		
				H. Seismic	-9.2	10.1	-7.6	-108	-185	269		
				0.300 Gravity	-1.7	0.1	-0.1	-3	-3	2		
		n=810		Total	-16.8	10.4	-8.1	-119	-198	278	0.61	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 809	237.0	Gravity	1.7	0.7	0.2	0	0	-29		
				H. Seismic	-15.6	6.9	3.9	1	0	-285		
				0.300 Gravity	-0.5	0.2	0.1	0	0	-9		
		n=810		Total	-14.4	7.8	4.2	1	0	-323	2.27	B14

Table 16. Forces, Moments, and DCRs (L11, 50-ft tower, no windows, $\theta_L = 90^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base (counteract)	S1	NPCM 12 n=102	134.1	Gravity H. Seismic 0.300 Gravity Total	-111.5 37.8 33.5 -40.2	1.5 103.3 0.4 105.2	-0.1 -6.9 0.0 -7.0	0 537 0 538	-5 -2430 -1 -2437	201 40165 60 40427	0.87	B1
Bent Column at Shaft (counteract)	S1	NPCM 44 n=538	160.1	Gravity H. Seismic 0.300 Gravity Total	-36.4 35.9 10.9 10.4	7.0 56.8 2.1 65.9	0.0 -2.5 0.0 -2.5	2 329 1 332	4 269 1 274	-652 -6827 -196 -7674	0.93	B2
Bent Beam at Shaft (counteract)	S1	TBNT 537 n=517	92.9	Gravity H. Seismic 0.300 Gravity Total	-5.7 17.5 1.7 13.5	-3.6 -43.6 -1.1 -48.3	0.0 -1.6 0.0 -1.6	1 291 0 292	1 348 0 350	-248 -6865 -74 -7188	1.69	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 538 n=531	-	Gravity H. Seismic 0.300 Gravity Total	-5.5 -14.8 -1.6 -22.0	-1.4 -38.4 -0.4 -40.3	0.0 3.0 0.0 3.1	1 291 0 292	0 0 0 0	0 0 0 0	0.42	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 511 n=518	-	Gravity H. Seismic 0.300 Gravity Total	-1.9 -15.9 -0.6 -18.4	16.1 13.7 4.8 34.7	-0.4 -4.4 -0.1 -4.9	1 3 0 4	0 0 0 0	0 0 0 0	0.41	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 6 n=533	37.9	Gravity H. Seismic 0.300 Gravity Total	-15.0 -6.5 -4.5 -26.1	-0.5 -23.1 -0.2 -23.8	0.1 2.4 0.0 2.6	3 92 1 95	-1 -39 0 -40	-31 -1657 -9 -1697	1.67	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 6 n=533	-	Gravity H. Seismic 0.300 Gravity Total	-15.0 6.5 4.5 -4.0	-0.5 -23.1 -0.2 -23.8	0.1 2.4 0.0 2.6	3 92 1 95	-1 -39 0 -40	-31 -1657 -9 -1697	10.9	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3 n=506	-	Gravity H. Seismic 0.300 Gravity Total	-9.0 4.7 2.7 -1.7	-0.2 -9.3 -0.1 -9.6	-1.6 -9.5 -0.5 -11.6	0 109 0 109	-72 -354 -22 -447	-49 -262 -15 -326	1.92	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 525 n=515	43.4	Gravity H. Seismic 0.300 Gravity Total	0.5 23.1 0.2 23.8	15.3 6.6 4.6 26.5	0.1 2.5 0.0 2.6	-1 -39 0 -40	-2 -91 -1 -93	-688 -1857 -207 -2752	0.98	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602 n=604	63.0	Gravity H. Seismic 0.300 Gravity Total	1.1 15.6 0.3 17.0	-2.7 -1.4 -0.8 -4.9	0.3 1.0 0.1 1.4	0 1 0 1	-18 -62 -5 -85	191 86 57 335	0.69	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 701 n=702	192.0	Gravity H. Seismic 0.300 Gravity Total	-2.9 -6.9 -0.9 -10.6	0.0 0.3 0.0 0.3	-0.1 0.0 0.0 -0.1	0 0 0 0	0 0 0 0	-4 -30 -1 -36	0.79	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2 n=704	90.4	Gravity H. Seismic 0.300 Gravity Total	0.3 -0.5 -0.1 -0.3	0.0 0.0 0.0 0.1	0.0 0.0 0.0 0.0	0 0 0 0	0 0 0 0	0 -2 0 -3	0.05	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 9 n=810	26.7	Gravity H. Seismic 0.300 Gravity Total	-5.8 -3.2 -1.7 -10.7	0.9 4.8 0.3 6.0	0.5 7.3 0.2 8.0	1 76 0 77	14 184 4 202	24 129 7 160	0.50	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 808 n=808	237.0	Gravity H. Seismic 0.300 Gravity Total	0.0 -8.6 0.0 -8.6	0.1 1.7 0.0 1.9	0.0 0.3 0.0 0.3	0 0 0 0	0 0 0 0	15 140 4 159	1.05	B14

San Carlos Tower – Cab Members and Connections

The Type L tower cab members and connections were most critical at the San Carlos tower. This tower cab was evaluated with analysis Cases L5 through L10, and L12 (see Table 6 for a summary). The most critical case was L6. Case L7 is for one window acting as a shear wall and cases L8 through L10 are for all the windows acting as shear walls. Only deflection results are presented for Cases L9 and L10, because these were never a critical load condition. The greatest problem in the cab evaluation is the excessive story drifts. The following sections summarize the Type L cab evaluations in terms of modal behavior, deflections, story drifts, and component forces and moments used to calculate DCRs.

Modal Analysis Results and Deflections

Table 17 gives the primary modes of vibration for the Type L, Cases L5 and L6 evaluations. Table 18 gives the maximum deflections at each floor level and other key locations in the San Carlos tower. Note that the deflections above the shaft roof become excessive. These are greater than any of the Salinas tower cases. Table 19 gives the greatest San Carlos story drifts, which are always for Case L6 in the cab. The calculated story drifts are the difference in SRSS deflections between floor levels (e.g., cab floor and roof). Table 19 shows that the story drift between the cab floor and roof is 14.14 in., which is more than four times greater than acceptable drifts calculated based on FEMA 302 guidance (Equation 24 of this report). These excessive drifts are due to very large rotations at the connections of the mullion base to the supporting beams at the shaft roof. The supporting shaft roof members provide little resistance to rotation. When seismic loads are applied along the Y-axis, four of the wide flange beams supporting the mullions are loaded in torsion, and these sections have very low torsional stiffness. These open sections (16WF40 and 10WF72) have very small polar moments of inertia, J . The fifth mullion is supported at the end of a cantilevered wide flange beam, and this also provides little rotational resistance. Connections of the mullions at the cab floor and roof are pinned (also connections above and below the windows), so that the only overturning resistance for the mullions is the very flexible moment connection at the mullion base. In reality, the shear connections at the cab floor and roof levels will provide some moment resistance, and this could have been modeled by adding rotational spring elements to the SAP2000 models. Even a very small amount of moment resistance at these joints greatly reduces deflections and explains why large deflections in the cab do not occur during strong winds or low seismic motions. However, this moment resistance should not properly be accounted for in the design or evaluation of these towers, and they remain very vulnerable to excessive deflections. This is the greatest Type L vulnerability that must be eliminated in upgrade development.

Table 17. San Carlos (Type L – 30 ft) Cab Evaluation Modal Analysis Results.

Mode #	L5 and L6				Mode of Vibration	L7 Period (sec)
	Period (sec)	Cumulative Participating Mass (%)				
		X-dir	Y-dir	Z-dir		
1	1.317	5.0	6.0	0.0	1 st YX-Lat. & Torsion/cab	1.307
2	1.273	8.3	15.5	0.1	1 st Y-Lateral/cab	0.973
3	0.776	14.4	15.5	0.1	1 st X-Lateral & Torsion/cab	0.448
6	0.271	15.4	15.6	0.1	2 nd Torsion	0.255
9	0.208	15.5	22.0	13.5	1 st Vertical & Y-Lateral	0.208
10	0.195	15.8	39.5	16.2	2 nd Y-Lateral	
11	0.189	45.4	39.6	16.3	2 nd X-Lateral	0.187
13	0.176	52.0	39.6	16.3	2 nd X-Lateral/windows	0.175
14	0.172	52.1	53.3	16.8	2 nd Y-Lateral/windows	0.167
15	0.151	52.2	53.9	17.0	Vertical/X-cab rocking	0.144
20	0.134	71.4	56.4	18.7	3 rd X-Lateral	0.127
21	0.128	72.1	57.0	20.8	Vertical/Y-cab rocking	0.128
22	0.122	72.5	71.8	20.9	3 rd Y-Lateral	0.097
30	0.097	73.4	76.9	31.7	3 rd Vertical/2 nd floor	0.097
34	0.089	73.5	76.9	42.6	4 th Vertical/shaft roof	0.089
49	0.062	77.2	77.1	47.4	3 rd Torsion	0.061
63	0.037	81.7	93.2	50.0	4 th Y-Lateral	0.037
64	0.035	94.1	96.3	50.0	4 th X-Lateral	0.035

Table 18. San Carlos Cab Evaluation Selected Horizontal SRSS Deflections.

Location	L5		L6		L7		L8		L9		L10	
	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	201	0.09	209	0.06	204	0.11	209	0.10	201	0.14	201	0.16
Shaft Roof, δ_2	501	0.34	513	0.24	508	0.42	513	0.38	501	0.51	501	0.60
Cab Floor, δ_3	609	2.49	609	3.11	609	2.72	607	1.56	609	1.64	609	1.39
Bottom of Windows	708	5.48	708	6.93	710	6.00	706	2.51	708	2.47	708	2.10
Top of Windows	814	11.93	814	15.13	818	13.18	810	3.26	814	3.13	814	2.63
Cab Roof, δ_4	916	13.58	916	17.25	921	14.97	911	3.51	916	3.32	916	2.78

Table 19. San Carlos Tower Story Drift and P-delta Effect Evaluation.

Location	Story	Story	Allow	Calc				Gravity	Seismic	Story	
	Elev	Height	Story	Analysis		Elastic	Story	Load	Shear	Stability	Drift w/
	Y_x	$h_{sx} =$	Drift	Case	Joint	Defl	Drift	above Y_x	Force	Coeff	P-delta
	(in.)	$Y_x - Y_{x-1}$	Δ_a		#	δ_x	Δ_c	P_x	V_x	θ	Δ_c
		(in.)	(in.)			(in.)	(in.)	(kips)	(kips)		(in.)
Ground Floor	0			L5	101	0.00					
2nd Floor	134	134	2.68	L5	201	0.09	0.09				
2nd Floor	134	134	2.68	L6	209	0.06					
Shaft Roof	294	160	3.2	L6	513	0.24	0.18				
Cab Floor	332	38	0.76	L6	609	3.11	2.87	60	100	0.046	2.87
Cab Roof	489	157	3.14	L6	916	17.25	14.14	26	63	0.038	14.14

Cab Member and Connection Evaluation

Tables 20 through 23 present the force and moment summary for the most critically stressed cab members and connections. These tables also present DCRs for each of these critical components. Case L6 with the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90^\circ$) gives the worst DCRs for cab members and connections. The mullion base plate connection fails with a very high DCR of 14.3. This failure is due to bending in the base plate for the smaller mullions (Col 1–4 on Drawing S3; Corlett & Spackman 1966). All other DCRs are much smaller. This failure was further evaluated by determining the seismic factor, F_1 , at which all mullion connections (including the heavier Col 5) have a DCR of 1.0. Table 24 summarizes the forces, moments, and DCRs for all the mullion connections to the tower shaft. The heavy Col 5 mullion connection fails by tension in the bolts. Table 25 shows the forces and moments at a seismic factor, F_1 equal to 0.1055, which gives an average DCR of 1.0. At this load condition, all the base plates will begin to yield (or bolt yielding for Col 5), and a hinge will form at each connection. This will cause a collapse mechanism, because no other connection to the mullions prevents mullion overturning. In reality, the shear connections will carry some moment so that yielding of the plates does not occur until a greater seismic factor. But these shear connectors will certainly not prevent collapse of the tower cab. The same upgrade that decreases story drift must also correct this connection failure.

Table 20. Forces, Moments, and DCRs (L5, 30-ft tower, no windows, $\theta_L = 0^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 32	134.3	Gravity	-65.5	-2.1	0.1	3	16	21		
				H. Seismic	15.6	-36.6	6.8	537	1515	8590		
				0.314 Gravity	20.6	-0.7	0.0	1	5	7		
(counteract)		n=118		Total	-29.3	-39.4	6.9	541	1536	8618	0.40	B1
Bent Column at Shaft	S1	NPCM 42	160.2	Gravity	-27.3	8.4	0.0	-1	-17	-724		
				H. Seismic	15.5	32.8	5.9	-286	-225	-1114		
				0.314 Gravity	8.6	2.6	0.0	0	-5	-227		
(counteract)		n=536		Total	-3.2	43.8	5.9	-288	-247	-2065	0.38	B2
Bent Beam at Shaft	S1	TBNT 532	92.9	Gravity	-6.3	-6.8	0.1	1	0	-438		
				H. Seismic	19.7	-8.2	2.2	241	334	-1324		
				0.314 Gravity	2.0	-2.1	0.0	0	0	-137		
(counteract)		n=508		Total	15.4	-17.1	2.3	242	335	-1899	0.56	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 536	-	Gravity	-4.8	-1.3	-0.1	-3	0	0		
				H. Seismic	-16.3	-10.1	-2.9	-243	0	0		
				0.314 Gravity	-1.5	-0.4	0.0	-1	0	0		
		n=531		Total	-22.6	-11.9	-3.0	-246	0	0	0.32	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 508	-	Gravity	-2.4	14.1	0.3	-1	0	0		
				H. Seismic	-10.2	13.1	5.4	-2	0	0		
				0.314 Gravity	-0.7	4.4	0.1	0	0	0		
		n=512		Total	-13.3	31.7	5.8	-4	0	0	0.37	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 3	37.9	Gravity	-8.9	-0.2	-1.6	0	-12	-41		
				H. Seismic	-4.3	-6.1	-11.7	-164	-429	-580		
				0.314 Gravity	-2.8	-0.1	-0.5	0	-4	-13		
		n=603		Total	-16.0	-6.4	-13.9	-164	-445	-634	1.35	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 5	-	Gravity	-12.5	1.5	0.0	3	-11	84		
				H. Seismic	6.3	24.1	-5.9	131	-56	1323		
				0.314 Gravity	3.9	0.5	0.0	1	-3	26		
		n=532		Total	-2.3	26.1	-6.0	136	-70	1434	9.49	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
				H. Seismic	4.3	-6.1	-11.7	-164	-823	-599		
				0.314 Gravity	2.8	-0.1	-0.5	0	-23	-16		
		n=506		Total	-1.9	-6.4	-13.9	-164	-918	-664	3.93	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 525	43.4	Gravity	0.5	15.3	0.1	-1	-2	-690		
				H. Seismic	21.0	5.4	5.1	-46	-210	-1559		
				0.314 Gravity	0.2	4.8	0.0	0	-1	-217		
		n=515		Total	21.6	25.5	5.3	-47	-213	-2465	0.96	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602	63.0	Gravity	1.1	-2.7	0.3	0	-18	191		
				H. Seismic	10.2	-1.1	0.8	1	-49	72		
				0.314 Gravity	0.4	-0.9	0.1	0	-6	60		
		n=604		Total	11.7	-4.7	1.2	1	-74	323	0.57	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 708	192.0	Gravity	-1.9	0.0	-0.1	0	0	-4		
				H. Seismic	-5.0	1.2	0.0	0	0	-113		
				0.314 Gravity	-0.6	0.0	0.0	0	0	-1		
		n=709		Total	-7.5	1.2	-0.1	0	0	-119	1.57	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 4	90.4	Gravity	0.4	0.0	0	0	0	0		
				H. Seismic	-0.5	0.0	0	0	0	-2		
				0.314 Gravity	-0.1	0.0	0	0	0	0		
		n=816		Total	-0.2	0.1	0	0	0	-3	0.05	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 6	26.7	Gravity	-5.3	-0.6	-0.3	0	-7	-15		
				H. Seismic	-5.1	-8.6	-7.6	95	-188	-231		
				0.314 Gravity	-1.7	-0.2	-0.1	0	-2	-5		
		n=801		Total	-12.0	-9.4	-8.0	95	-197	-251	0.57	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 819	237.0	Gravity	-0.5	-0.2	0.0	0	0	18		
				H. Seismic	-13.0	-1.7	-0.3	0	0	139		
				0.314 Gravity	-0.2	-0.1	0.0	0	0	5		
		n=820		Total	-13.7	-1.9	-0.3	0	0	162	1.25	B14

Table 21. Forces, Moments, and DCRs (L6, 30-ft tower, no windows, $\theta_L = 90^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 33	134.3	Gravity	-74.6	-0.2	0.1	1	2	121		
				H. Seismic	18.3	-35.0	4.1	342	995	7148		
				0.314 Gravity	23.4	0.0	0.0	0	1	38		
(counteract)		n=119		Total	-32.9	-35.2	4.3	343	998	7306	0.29	B1
Bent Column at Shaft	S1	NPCM 42	160.2	Gravity	-27.3	8.4	0.0	-1	-17	-724		
				H. Seismic	12.9	30.7	3.9	-193	-159	-1303		
				0.314 Gravity	8.6	2.6	0.0	0	-5	-227		
(counteract)		n=536		Total	-5.9	41.8	4.0	-194	-181	-2254	0.35	B2
Bent Beam at Shaft	S1	TBNT 532	92.9	Gravity	-6.3	-6.8	0.1	1	0	-438		
				H. Seismic	22.8	-9.3	2.0	162	229	-1495		
				0.314 Gravity	2.0	-2.1	0.0	0	0	-137		
(counteract)		n=508		Total	18.5	-18.2	2.1	163	230	-2070	0.58	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 538	-	Gravity	-6.3	-1.6	0.0	1	0	0		
				H. Seismic	-18.2	-11.3	2.2	164	0	0		
				0.314 Gravity	-2.0	-0.5	0.0	0	0	0		
		n=531		Total	-26.5	-13.3	2.2	165	0	0	0.37	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 511	-	Gravity	-2.2	16.3	-0.4	1	0	0		
				H. Seismic	-7.5	16.9	-5.8	3	0	0		
				0.314 Gravity	-0.7	5.1	-0.1	0	0	0		
		n=518		Total	-10.3	38.3	-6.3	5	0	0	0.45	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 6	37.9	Gravity	-15.1	-0.5	0.1	3	-1	-31		
				H. Seismic	-12.0	-28.5	2.6	129	-56	-1954		
				0.314 Gravity	-4.7	-0.2	0.0	1	0	-10		
		n=533		Total	-31.8	-29.2	2.8	133	-57	-1994	1.98	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 6	-	Gravity	-15.1	-0.5	0.1	3	-1	-31		
				H. Seismic	12.0	-28.5	2.6	129	-56	-1954		
				0.314 Gravity	4.7	-0.2	0.0	1	0	-10		
		n=533		Total	1.7	-29.2	2.8	133	-57	-1994	14.33	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
				H. Seismic	6.1	-8.2	-8.3	-142	-314	-230		
				0.314 Gravity	2.8	-0.1	-0.5	0	-23	-16		
		n=506		Total	-0.1	-8.5	-10.4	-142	-409	-295	1.77	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 525	43.4	Gravity	0.5	15.3	0.1	-1	-2	-690		
				H. Seismic	28.5	12.1	2.6	-56	-86	-2134		
				0.314 Gravity	0.2	4.8	0.0	0	-1	-217		
		n=515		Total	29.2	32.2	2.8	-57	-88	-3040	1.09	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602	63.0	Gravity	1.1	-2.7	0.3	0	-18	191		
				H. Seismic	13.2	-1.9	1.0	1	-62	119		
				0.314 Gravity	0.4	-0.9	0.1	0	-6	60		
		n=604		Total	14.7	-5.5	1.4	1	-87	370	0.66	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 701	192.0	Gravity	-2.9	0.0	-0.1	0	0	-4		
				H. Seismic	-7.3	1.2	0.0	0	0	-111		
				0.314 Gravity	-0.9	0.0	0.0	0	0	-1		
		n=702		Total	-11.0	1.2	-0.1	0	0	-117	1.84	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2	90.4	Gravity	0.3	0.0	0.0	0	0	0		
				H. Seismic	-0.2	0.0	0.0	0	0	-2		
				0.314 Gravity	-0.1	0.0	0.0	0	0	0		
		n=807		Total	0.0	0.1	0.0	0	0	-3	0.04	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 7	26.7	Gravity	-4.4	-1.0	0.3	4	6	-27		
				H. Seismic	-7.1	-11.7	6.6	143	147	-312		
				0.314 Gravity	-1.4	-0.3	0.1	1	2	-8		
		n=805		Total	-12.9	-13.0	6.9	148	155	-348	0.59	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 808	237.0	Gravity	0.0	0.1	0.0	0	0	15		
				H. Seismic	-9.3	1.9	0.3	0	0	153		
				0.314 Gravity	0.0	0.0	0.0	0	0	5		
		n=808		Total	-9.3	2.1	0.3	0	0	173	1.15	B14

Table 22. Forces, Moments, and DCRs (L7, 30-ft tower, one window, $\theta_L = 55.3^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 33	134.3	Gravity	-75.0	-0.2	0.1	1	1	132		
				H. Seismic	21.7	-41.9	9.0	799	1953	9178		
				0.314 Gravity	23.6	-0.1	0.0	0	0	41		
(counteract)		n=119		Total	-29.8	-42.2	9.1	801	1954	9351	0.48	B1
Bent Column at Shaft	S1	NPCM 41	160.2	Gravity	-30.7	7.8	0.0	1	20	-755		
				H. Seismic	15.3	33.4	-7.9	512	333	-1037		
				0.314 Gravity	9.6	2.4	0.0	0	6	-237		
(counteract)		n=535		Total	-5.7	43.6	-7.9	514	359	-2030	0.37	B2
Bent Beam at Shaft	S1	TBNT 532	92.9	Gravity	-6.2	-7.1	0.2	0	3	-480		
				H. Seismic	21.4	-6.8	2.4	374	474	-997		
				0.314 Gravity	1.9	-2.2	0.1	0	1	-151		
(counteract)		n=508		Total	17.2	-16.1	2.6	375	478	-1628	0.54	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 536	-	Gravity	-5.0	-1.2	0.0	-3	0	0		
				H. Seismic	-19.6	-11.7	-4.2	-374	0	0		
				0.314 Gravity	-1.6	-0.4	0.0	-1	0	0		
		n=531		Total	-26.2	-13.3	-4.3	-377	0	0	0.37	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 508	-	Gravity	-2.7	14.7	0.3	-1	0	0		
				H. Seismic	-9.3	20.4	6.0	-3	0	0		
				0.314 Gravity	-0.8	4.6	0.1	0	0	0		
		n=512		Total	-12.8	39.6	6.5	-5	0	0	0.47	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 4	37.9	Gravity	-13.9	3.2	-0.3	-4	8	112		
				H. Seismic	-7.6	19.0	-4.5	-137	76	1097		
				0.314 Gravity	-4.4	1.0	-0.1	-1	3	35		
		n=506		Total	-25.9	23.2	-4.9	-142	87	1244	1.33	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 5	-	Gravity	-13.3	1.4	-0.4	-1	-7	116		
				H. Seismic	15.9	28.8	-4.0	-149	-38	1594		
				0.314 Gravity	4.2	0.4	-0.1	0	-2	36		
		n=532		Total	6.7	30.6	-4.5	-150	-48	1746	14.06	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3	-	Gravity	-8.2	0.4	-2.0	-15	-53	-35		
				H. Seismic	13.8	4.4	-11.5	-150	-571	-416		
				0.314 Gravity	2.6	0.1	-0.6	-5	-17	-11		
		n=506		Total	8.2	4.9	-14.1	-170	-641	-463	2.92	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 523	30.8	Gravity	-1.4	13.5	-0.4	-7	10	-298		
				H. Seismic	-28.8	15.9	-4.1	-37	125	-1712		
				0.314 Gravity	-0.4	4.2	-0.1	-2	3	-93		
		n=511		Total	-30.6	33.7	-4.5	-47	138	-2103	0.80	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602	63.0	Gravity	1.7	-2.7	0.2	0	-14	191		
				H. Seismic	8.6	-1.4	0.6	1	-39	85		
				0.314 Gravity	0.5	-0.9	0.1	0	-4	60		
		n=603		Total	10.9	-5.0	0.9	1	-57	336	0.52	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 706	192.0	Gravity	-1.8	0.0	-0.1	0	0	-4		
				H. Seismic	-7.0	1.2	0.0	0	0	-115		
				0.314 Gravity	-0.6	0.0	0.0	0	0	-1		
		n=707		Total	-9.3	1.2	-0.1	0	0	-121	1.74	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2	90.4	Gravity	0.4	0.0	0	0	0	0		
				H. Seismic	-7.7	0.0	0	-2	0	0		
				0.314 Gravity	-0.1	0.0	0	0	0	0		
		n=807		Total	-7.5	0.0	0	-2	0	0	0.41	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 9	26.7	Gravity	-5.8	0.3	-0.5	-9	-10	7		
				H. Seismic	-8.4	9.5	-9.2	-98	-227	254		
				0.314 Gravity	-1.8	0.1	-0.1	-3	-3	2		
		n=810		Total	-16.1	9.8	-9.8	-110	-240	263	0.66	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 809	237.0	Gravity	1.7	0.7	0.2	0	0	-29		
				H. Seismic	-12.4	5.1	2.9	1	0	-211		
				0.314 Gravity	-0.5	0.2	0.1	0	0	-9		
		n=810		Total	-11.2	6.0	3.2	1	0	-249	1.66	B14

Table 23. Forces, Moments, and DCRs (L8, 30-ft tower, all windows, $\theta_L = 90^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base (counteract)	S1	NPCM 33 n=119	134.3	Gravity H. Seismic 0.314 Gravity Total	-74.8 27.9 23.5 -23.5	-0.2 -44.1 -0.1 -44.4	0.1 7.1 0.0 7.2	1 615 0 617	2 1693 1 1696	139 9359 44 9542	0.46	B1
Bent Column at Shaft (counteract)	S1	NPCM 42 n=536	160.2	Gravity H. Seismic 0.314 Gravity Total	-27.1 28.2 8.5 9.7	9.2 38.0 2.9 50.1	0.0 6.3 0.0 6.3	-2 -318 -1 -321	-16 -224 -5 -244	-787 -1385 -247 -2420	0.43	B2
Bent Beam at Shaft (counteract)	S1	TBNT 532 n=508	92.9	Gravity H. Seismic 0.314 Gravity Total	-6.3 33.5 2.0 29.2	-7.3 -9.1 -2.3 -18.7	0.2 1.7 0.0 1.9	0 289 0 290	1 359 0 360	-513 -1508 -161 -2183	0.66	B3
Bent conn at center of tower roof (additive)	S1/2,3	TBNT 538 n=531	-	Gravity H. Seismic 0.314 Gravity Total	-7.5 -6.4 -2.4 -16.3	-1.2 -9.4 -0.4 -10.9	0.0 -3.3 0.0 -3.3	0 290 0 291	0 0 0 0	0 0 0 0	0.23	B4
Bent/roof beam conn 16WF88 (additive)	S2 S4/13,19 S5/17,18	EB 509 n=514	-	Gravity H. Seismic 0.314 Gravity Total	-3.3 -12.2 -1.0 -16.4	14.6 23.1 4.6 42.4	-0.3 -3.0 -0.1 -3.3	2 5 1 7	0 0 0 0	0 0 0 0	0.49	B5
Corner mullions at their base (additive)	S4/A,2,3 col. 1-4	CLCM 3 n=603	37.9	Gravity H. Seismic 0.314 Gravity Total	-8.0 -21.9 -2.5 -32.4	0.8 15.8 0.3 16.9	-2.6 -9.2 -0.8 -12.6	-2 -29 -1 -31	4 183 1 189	-99 -652 -31 -783	1.09	B6
Corner mullion/ 10WF conn (counteract)	S4/A,2,5 col. 1-4	CLCM 6 n=533	-	Gravity H. Seismic 0.314 Gravity Total	-15.9 27.1 4.8 16.0	1.8 16.5 0.5 18.8	0.1 4.1 0.0 4.2	1 20 0 21	0 18 0 18	147 565 44 756	7.86	B7
Corner mullion/ 10WF conn (counteract)	S4/8,9 col. 5	CLCM 3 n=506	-	Gravity H. Seismic 0.314 Gravity Total	-8.2 21.9 2.6 16.3	0.8 15.8 0.3 16.9	-2.6 -9.2 -0.8 -12.6	-2 -29 -1 -31	-95 -241 -30 -366	-68 -171 -21 -260	1.76	B8
10WF72 at 16WF88 shaft roof (additive)	S2,S3 S4/A	RAD 525 n=515	43.4	Gravity H. Seismic 0.314 Gravity Total	-1.8 -16.5 -0.6 -18.9	16.2 27.3 5.1 48.6	0.1 4.1 0.0 4.2	0 18 0 18	-3 -197 -1 -201	-550 -1576 -173 -2298	0.90	B9
Cab floor 12WF27 near Col 5 (additive)	S2	CEPG 602 n=604	63.0	Gravity H. Seismic 0.314 Gravity Total	2.5 9.6 0.8 12.9	-2.7 -4.6 -0.9 -8.2	0.2 0.6 0.1 0.8	0 0 0 0	-11 -35 -3 -49	191 288 60 539	0.77	B10
C 4 x 7.25 at base of window (additive)	S3	CEPG 703 n=703	192.0	Gravity H. Seismic 0.314 Gravity Total	-0.8 -14.6 -0.2 -15.6	0.0 1.8 0.0 1.8	-0.2 -12.0 0.0 -12.1	0 0 0 0	0 0 0 0	-1 -87 0 -88	1.88	B11
Interior mullion (S3x7.5) (buckling)	A5/5	WM 2 n=807	90.4	Gravity H. Seismic 0.314 Gravity Total	0.5 -10.6 -0.1 -10.3	0.0 0.0 0.0 0.0	0 0 0 0	0 0 0 1	0 0 0 0	0 0 0 -1	0.58	B12
Corner mullion at the roof (additive)	S4/1,2,3,7	CUCM 9 n=810	26.7	Gravity H. Seismic 0.314 Gravity Total	-2.6 -9.5 -0.8 -12.9	0.3 1.9 0.1 2.3	0.2 5.1 0.1 5.3	5 50 2 57	4 147 1 152	9 51 3 63	0.33	B13
C 6 x 8.2 at top of window (buckling)	S3	CEPG 809 n=810	237.0	Gravity H. Seismic 0.314 Gravity Total	0.8 -7.9 -0.3 -7.4	0.7 6.2 0.2 7.1	0.2 3.7 0.0 3.9	0 0 0 -1	0 0 0 0	-26 -257 -8 -292	1.85	B14

Table 24. Corner Mullion Base Connection Evaluation - Forces, Moments, and DCRs (L6, 30-ft tower, no windows, $\theta_s = 90^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 1 n=503	-	Gravity H. Seismic 0.314 Gravity Total	-10.5 6.1 3.3 -1.1	-1.2 -14.5 -0.4 -16.1	1.6 14.1 0.5 16.3	3 131 1 134	83 374 26 483	-58 -267 -18 -343	2.33	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 3 12" COL n=506	-	Gravity H. Seismic 0.314 Gravity Total	-9.0 6.1 2.8 -0.1	-0.2 -8.2 -0.1 -8.5	-1.6 -8.3 -0.5 -10.4	0 -142 0 -142	-73 -314 -23 -409	-49 -230 -16 -295	2.04	B8
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 4 n=534	-	Gravity H. Seismic 0.314 Gravity Total	-13.9 8.8 4.4 -0.8	2.2 26.1 0.7 29.0	-0.1 -3.9 0.0 -4.0	0 146 0 146	9 69 3 81	68 1623 21 1711	11.67	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 5 n=532	-	Gravity H. Seismic 0.314 Gravity Total	-12.5 7.9 3.9 -0.7	1.5 24.3 0.5 26.3	0.0 -3.8 0.0 -3.8	3 148 1 153	-11 -83 -3 -98	84 1373 26 1483	10.13	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 6 n=533	-	Gravity H. Seismic 0.314 Gravity Total	-15.1 12.0 4.7 1.6	-0.5 -28.5 -0.2 -29.2	0.1 2.6 0.0 2.7	3 129 1 133	-1 -56 0 -57	-31 -1954 -10 -1995	14.31	B7
									Average DCR = 8.09			

Table 25. Corner Mullion Base Connection Evaluation - Forces, Moments, and DCRs (Seismic Factor, $F_s = 0.1055$, L6, 30-ft tower, no windows, $\theta_i = 90^\circ$, $S_{DS} = 0.113g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 1 n=503	-	Gravity H. Seismic 0.314 Gravity Total	-10.5 0.6 0.3 -9.6	-1.2 -1.5 0.0 -2.8	1.6 1.5 0.0 3.1	3 13.8 0.1 16	83 39.4 2.0 124	-58 -28.1 -1.4 -87	0.55	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 3 12" COL n=506	-	Gravity H. Seismic 0.314 Gravity Total	-9.0 0.6 0.2 -8.2	-0.2 -0.9 0.0 -1.1	-1.6 -0.9 0.0 -2.5	0 -15.0 0.0 -15	-73 -33.1 -1.8 -107	-49 -24.3 -1.2 -75	0.48	B8
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 4 n=534	-	Gravity H. Seismic 0.314 Gravity Total	-13.9 0.9 0.3 -12.7	2.2 2.8 0.1 5.0	-0.1 -0.4 0.0 -0.5	0 15.4 0.0 16	9 7.3 0.2 17	68 171.1 1.6 240	1.32	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 5 n=532	-	Gravity H. Seismic 0.314 Gravity Total	-12.5 0.8 0.3 -11.4	1.5 2.6 0.0 4.1	0.0 -0.4 0.0 -0.4	3 15.6 0.1 19	-11 -8.8 -0.3 -20	84 144.8 2.1 231	1.29	B7
Corner mullion/ 10WF conn	S-4/A,2,5 S-4/8,9	CLCM 6 n=533	-	Gravity H. Seismic 0.314 Gravity Total	-15.1 1.3 0.4 -13.5	-0.5 -3.0 0.0 -3.5	0.1 0.3 0.0 0.4	3 13.6 0.1 17	-1 -5.9 0.0 -7	-31 -206.1 -0.8 -238	1.29	B7
									Average DCR = 0.99			

5 Type L Seismic Upgrade Development

Description of Proposed Upgrade for All Type L Towers

The upgrade approach proposed here consists of welding very deep structural tube (ST 20 x 4 x ½) members to the base of each mullion in a pentagon configuration (see Figure 11). These members must also be anchored to the horizontal portion of the concrete bents. This connection only needs to transfer shear forces, not moments. The tubes must be welded to the mullions and each other to form a continuous pentagon. This pentagon structure will form a very stiff foundation to greatly reduce rotations by itself. The tubes will also relieve the overstressed mullion connections to the shaft roof beams. These tubes will never allow enough rotation to significantly stress the existing mullion base plates and bolts. The tubes by themselves, however, will not bring the story drift down to acceptable levels. Figure 11 also shows the stiffening and strengthening of the mullions, by welding 5 in. x 1.5 in. plates on both faces of the mullions. This, together with the ST 20 x 4 x ½ tubes around the perimeter, significantly reduces cab deflections and DCRs in the mullions.

Salinas Tower Upgrade – Shaft Members and Connections

Analysis Case L13* was used to evaluate the application of the seismic upgrade described above to the Type L tower shaft at the 50-ft tall Salinas tower. Because the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90^\circ$) gave one of the worst-case loading for critical shaft elements (Case L1), the same seismic loading is used to evaluate the upgraded tower in Case L13.

* The San Carlos tower upgrade was first evaluated with the L12 analysis case because the upgrade was developed for the more vulnerable cab of the San Carlos tower.

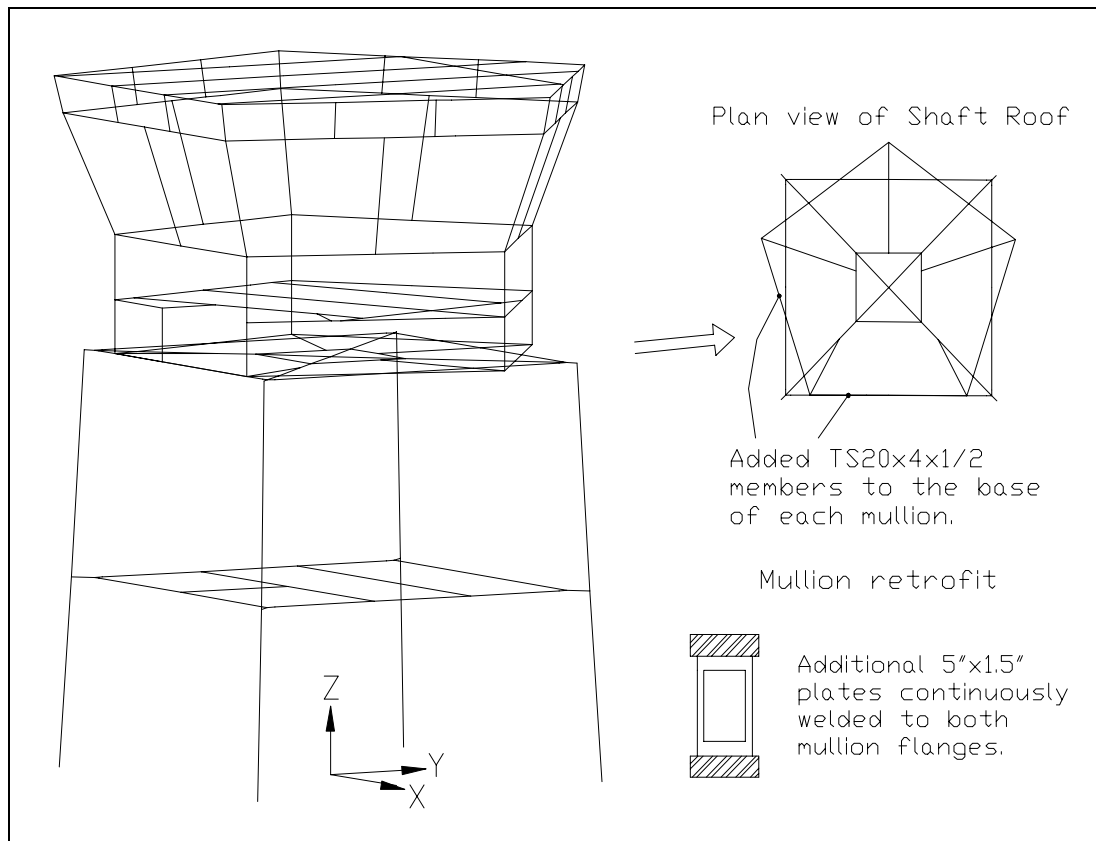


Figure 11. Finite element mesh for the upgraded 30-ft tall San Carlos tower.

Modal Analysis Results and Deflections

Tables 26, 27, and 28 summarize the 50-ft Salinas tower upgrade modal analysis results, deflections, and story drifts. Drifts fall well within allowable values.

Shaft Member and Connection Evaluation

Table 29 presents the force and moment summary for the most critically stressed members and connections. The column bent is slightly overstressed with a DCR of 2.21, but this is based on conservative modeling that will cause the bent to be more heavily loaded than reality. The DCRs for all other members and connections in the shaft fall below 2.0, demonstrating that the proposed upgrade is adequate for the tower shaft.

Table 26. Salinas (Type L – 50 ft) Shaft Upgrade Modal Analysis Results.

Mode #	L13				Mode of Vibration
	Period (sec)	Cumulative Participating Mass (%)			
		X-dir	Y-dir	Z-dir	
1	0.692	2.9	0.5	0.0	1 st Torsion
2	0.562	3.6	63.3	0.0	1 st Y-Lateral
3	0.542	65.8	63.8	0.0	1 st X-Lateral
4	0.249	65.8	68.1	1.2	2 nd Y-Lateral
5	0.227	66.9	68.2	1.2	2 nd Torsion
6	0.184	68.4	68.2	1.2	2 nd X-Lateral
7	0.147	68.4	68.9	10.5	1 st Vertical/cab floor & roof
8	0.142	69.1	68.9	10.5	3 rd Torsion
9	0.129	69.2	69.6	11.3	2 nd Vertical/cab roof & floor
10	0.114	82.3	69.6	11.4	3 rd X-Lateral/cab rocking
11	0.113	82.4	83.2	11.4	3 rd Y-Lateral/cab rocking
15	0.097	84.0	84.3	19.5	3 rd Vertical/4 th floor & roof
18	0.094	84.5	84.4	30.6	4 th Vertical/2 nd & 3 rd floor
28	0.077	85.1	85.3	38.2	4 th Torsion
48	0.051	86.1	89.6	47.7	4 th Y-Lateral
49	0.049	87.7	89.6	47.9	4 th X-Lateral
50	0.048	89.6	90.9	48.0	4 th X-Lateral
51	0.047	90.9	90.9	48.0	4 th X-Lateral

Table 27. Salinas Shaft Upgrade Selected Horizontal SRSS Deflections.

Location	L13	
	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	204	0.27
3 rd Floor, δ_2	304	1.11
4 th Floor, δ_3	404	2.41
Shaft Roof, δ_4	508	3.92
Cab Floor, δ_5	607	4.23
Bottom of Windows	706	4.63
Top of Windows	810	5.11
Cab Roof, δ_6	911	5.28

Table 28. Salinas Tower Upgrade Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Calc Elastic Story Drift			
				Analysis Case	Joint #	Defl δ_x (in.)	Drift Δ_c (in.)
Ground Floor	0			L13	102	0.00	
2nd Floor	134	134	2.68	L13	204	0.27	0.27
3rd Floor	278	144	2.88	L13	304	1.11	0.84
4th Floor	422	144	2.88	L13	404	2.41	1.30
Shaft Roof	582	160	3.2	L13	508	3.92	1.51
Shaft Roof	582	160	3.2	L13	513	3.87	
Cab Floor	620	38	0.76	L13	607	4.23	0.37
Cab Roof	777	157	3.14				

Table 29. Salinas Upgrade Forces, Moments, and DCRs (L13, 50-ft tower, all windows, $\theta_L = 90^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 12	134.1	Gravity	-113.7	1.3	0.0	0	-5	202		
				H. Seismic	78.6	117.4	-7.7	-574	-3227	47246		
				0.300 Gravity	34.1	0.4	0.0	0	-1	61		
(counteract)		n=102		Total	-1.0	119.1	-7.7	-574	-3233	47508	1.10	B1
Bent Column at Shaft	S1	NPCM 42	160.1	Gravity	-29.4	12.8	-0.3	2	16	-1315		
				H. Seismic	78.1	75.7	-2.8	349	343	-8456		
				0.300 Gravity	8.8	3.8	-0.1	1	5	-395		
(counteract)		n=536		Total	57.5	92.4	-3.2	351	364	-10166	1.30	B2
Bent Beam at Shaft	S1	TBNT 553	92.9	Gravity	-8.8	-22.5	-0.2	2	4	-971		
				H. Seismic	44.1	-47.4	-13.6	347	316	-8138		
				0.300 Gravity	2.6	-6.8	-0.1	1	1	-291		
(counteract)		n=508		Total	38.0	-76.7	-13.9	349	321	-9401	2.21	B3
Bent conn at center of tower roof	S1/2,3	TBNT 538	-	Gravity	-8.5	-0.4	0.0	1	0	0		
				H. Seismic	-8.4	-50.3	-3.4	276	0	0		
				0.300 Gravity	-2.6	-0.1	0.0	0	0	0		
(additive)		n=531		Total	-19.5	-50.8	-3.4	277	0	0	0.53	B4
Bent/roof beam conn	S2	EB 505	-	Gravity	-3.7	0.6	0.0	0	0	0		
	S4/13,19			H. Seismic	-33.2	42.2	1.2	1	0	0		
	S5/17,18			0.300 Gravity	-1.1	0.2	0.0	0	0	0		
(additive)		n=507		Total	-37.9	43.0	1.2	1	0	0	0.50	B5
Corner mullions at their base	S4/A,2,3	CLCM 4	37.9	Gravity	-14.2	12.5	6.5	0	208	453		
				H. Seismic	-7.9	9.5	37.5	-15	1251	281		
				0.300 Gravity	-4.3	3.8	2.0	0	62	136		
(additive)	col. 1-4	n=534		Total	-26.3	25.8	46.0	-15	1521	870	1.51	B6a
Corner mullion/ 10WF conn	S4/A,2,5	CLCM 4	-	Gravity	-14.2	12.5	6.5	0	0	0		
				H. Seismic	7.9	9.5	37.5	-15	0	0		
				0.300 Gravity	4.3	3.8	2.0	0	0	0		
(counteract)	col. 1-4	n=534		Total	-2.1	25.8	46.0	-15	0	0	0.83	B7
Corner mullion/ 10WF conn	S4/8,9	CLCM 3	-	Gravity	-9.4	4.9	-4.4	-1	0	0		
				H. Seismic	21.8	16.6	-7.3	-52	0	0		
				0.300 Gravity	2.8	1.5	-1.3	0	0	0		
(counteract)	col. 5	n=506		Total	15.2	23.0	-13.0	-53	0	0	0.31	B8
10WF72 at 16WF88 shaft roof	S2,S3 S4/A	RAD 525	43.4	Gravity	-0.2	9.9	0.0	0	0	-261		
				H. Seismic	-3.8	17.8	2.5	0	74	-898		
				0.300 Gravity	-0.1	3.0	0.0	0	0	-78		
(additive)		n=515		Total	-4.1	30.7	2.5	0	74	-1237	0.46	B9
Cab floor 12WF27 near Col 5	S2	CEPG 602	63.0	Gravity	5.8	-2.7	0.0	0	0	191		
				H. Seismic	11.7	-2.1	-0.2	0	15	129		
				0.300 Gravity	1.7	-0.8	0.0	0	0	57		
(additive)		n=604		Total	19.3	-5.6	-0.2	0	15	378	0.49	B10
C 4 x 7.25 at base of window	S3	CEPG 705	192.0	Gravity	-0.9	0.2	0.0	0	0	-7		
				H. Seismic	-6.4	1.6	0.7	0	0	-79		
				0.300 Gravity	-0.3	0.1	0.0	0	0	-2		
(additive)		n=706		Total	-7.6	1.9	0.6	0	0	-88	1.23	B11
Interior mullion (S3x7.5)	A5/5	WM 2	90.4	Gravity	0.2	0.0	0	0	0	0		
				H. Seismic	-11.5	0.0	0	0	0	0		
				0.300 Gravity	0.0	0.0	0	0	0	0		
(buckling)		n=807		Total	-11.4	0.0	0	0	0	0	0.63	B12
Corner mullion at the roof	S4/1,2,3,7	CUCM 6	26.7	Gravity	-2.6	-0.2	0.5	6	11	-4		
				H. Seismic	-1.6	-6.6	6.0	48	172	-175		
				0.300 Gravity	-0.8	-0.1	0.1	2	3	-1		
(additive)		n=801		Total	-4.9	-6.8	6.6	56	186	-180	0.24	B13
C 6 x 8.2 at top of window	S3	CEPG 809	237.0	Gravity	0.5	0.5	0.1	0	0	-20		
				H. Seismic	-8.7	6.3	3.9	0	0	-262		
				0.300 Gravity	0.1	0.2	0.0	0	0	-6		
(buckling)		n=810		Total	-8.1	7.0	4.0	0	0	-288	1.86	B14
TS 20x4x1/2 around mull. perimeter	NA	TUBE 557	70.7	Gravity	-14.5	-7.3	0.0	30	0	534		
				H. Seismic	-15.7	-13.3	0.4	63	28	940		
				0.314 Gravity	-4.6	-2.3	0.0	9	0	168		
(additive)	retrofit member	n=534		Total	-34.7	-22.9	0.4	102	28	1641	0.41	B15

San Carlos Tower Upgrade – Cab Members and Connections

Analysis Case L12 was used to evaluate the application of the seismic upgrade to the most critical Type L tower cab at the 30-ft tall San Carlos tower. Because the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90^\circ$) gave the worst case loading for critical cab elements (Case L6), the same seismic loading was used to evaluate the upgraded tower in Case L12.

Modal Analysis Results and Deflections

Tables 30, 31, and 32 summarize the 30-ft San Carlos tower upgrade modal analysis results, deflections, and story drifts. Calculated story drifts exceed allowable values by 50 percent. In an actual earthquake, these deflections would be reduced by the additional stiffening of the mullions by the deep structural tubing member that prevents deflections and rotations of the mullions up to the depth of the structural tubing (20 in.). Also, for a moment frame structure (for the upgraded configuration), FEMA 302 gives an R-value of 8 and a deflection amplification value, C_d , of 5.5. For design purposes, the effective story drift could be reduced to acceptable levels by dividing by R (8) and multiplying by C_d (5.5).

The addition of these tubes reduces rotations at the shaft roof mullion connections by more than an order of magnitude. Table 33 presents the rotations at the base of these mullions before (Case L6) and after the upgrade (Case L12). The difference in elevation between the cab roof and shaft roof in this model is 195 in. (777 – 582 in.). The rotations at the mullion connections to the shaft roof in Table 33 were multiplied by the 195-in. mullion height to give the deflections due to rotations alone. The SRSS deflections, δ for both the x and y rotations (R_x and R_y respectively), were calculated as follows:

$$\delta = 195 \text{ inches} \sqrt{R_x^2 + R_y^2} \quad [\text{Eq 26}]$$

These deflections are given in Table 33. This shows the influence of the rotations on deflections and the effectiveness of the proposed upgrade in reducing these deflections. The story drifts in Table 32 are greater than these values due to bending of the upgraded mullions.

Cab Member and Connection Evaluation

Table 34 summarizes the force and moment for the most critically stressed members and connections. The greatest DCR was 1.56 (below 2.0), demonstrating the effectiveness of the proposed upgrade. Table 34 shows that the upgraded

mullion has a DCR of only 1.47. The capacity of this member is defined in Appendix B6a and the applied loads in this appendix are those shown in Table 34 for the L12 analysis case. The bottom of Table 34 shows that the TS 20 x 4 x ½ structural tube used to upgrade this tower has a DCR of only 0.61, demonstrating its effective performance. The capacity of this member is defined in Appendix B15.

Table 30. San Carlos (Type L – 30 ft) Cab Upgrade Modal Analysis Results.

Mode #	L12				Mode of Vibration
	Period (sec)	Cumulative Participating Mass (%)			
		X-dir	Y-dir	Z-dir	
1	0.709	3.1	0.4	0.0	1 st Torsion/cab
2	0.559	4.8	13.8	0.1	1 st Y-Lateral/cab
3	0.539	16.5	17.0	0.1	1 st X-Lateral/cab
6	0.270	17.0	17.1	0.1	2 nd Torsion
9	0.183	17.0	27.8	9.7	1 st Vertical & Y-Lateral
10	0.179	17.0	28.7	12.2	2 nd Vertical/cab roof
13	0.166	73.1	28.8	12.3	2 nd X-Lateral
14	0.163	73.2	74.2	13.9	2 nd Y-Lateral
15	0.143	73.5	74.2	17.0	Vertical/X-cab rocking
26	0.097	75.5	75.3	30.0	3 rd Vertical/2 nd floor
33	0.083	75.5	75.7	43.0	4 th Vertical/shaft roof
47	0.061	75.9	75.8	46.0	3 rd Torsion
60	0.036	85.6	95.1	51.1	4 th Y-Lateral
61	0.034	98.7	97.5	51.1	4 th X-Lateral

**Table 31. San Carlos Cab Upgrade
Selected Horizontal SRSS Deflections.**

Location	L12	
	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	204	0.08
Shaft Roof, δ_2	508	0.33
Cab Floor, δ_3	607	0.66
Bottom of Windows	703	1.61
Top of Windows	805	4.52
Cab Roof, δ_4	907	5.40

Table 32. San Carlos Tower Upgrade Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Calc			
				Analysis Case	Joint #	Elastic Defl δ_x (in.)	Story Drift Δ_c (in.)
Ground Floor	0			L12			
2nd Floor	134	134	2.68	L12	204	0.08	0.08
Shaft Roof	294	160	3.2	L12	508	0.33	0.25
Cab Floor	332	38	0.76	L12	603	0.66	0.32
Cab Roof	489	157	3.14	L12	907	5.404	4.75

Table 33. Rotations at the Base of the Mullions Before and After the Upgrade.

Mullion (Col #*)	Joint Number	Case L6 Rotations and Deflections			Case L12 Rotations and Deflections		
		R_x (radians)	R_y (radians)	δ (in.)	R_x (radians)	R_y (radians)	δ (in.)
1	503	0.0451	0.0013	8.80	0.0049	0.0006	0.96
2	534	0.0506	0.0221	10.77	0.0009	0.0035	0.70
3	533	0.0184	0.0607	12.37	0.0064	0.0006	1.25
4	532	0.0613	0.0245	12.87	0.0013	0.0044	0.89
5	506	0.0546	0.0008	10.65	0.0052	0.0009	1.03

* Drawing S3 (Corlett & Spackman 1966).

Table 34. San Carlos Upgrade Forces, Moments, and DCRs (L12, 30-ft tower, no windows, $\theta_L = 90^\circ$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column at Base	S1	NPCM 32	134.3	Gravity	-67.6	-5.2	0.6	4	50	-62		
				H. Seismic	16.8	-54.1	5.0	314	1222	-1292		
				0.314 Gravity	21.2	-1.6	0.2	1	16	-19		
(counteract)		n=118		Total	-29.6	-60.9	5.7	319	1288	-1373	0.49	B1
Bent Column at Shaft	S1	NPCM 42	160.2	Gravity	-30.0	14.7	-0.4	2	18	-1318		
				H. Seismic	16.0	41.5	-4.3	195	294	-1335		
				0.314 Gravity	9.4	4.6	-0.1	1	6	-414		
(counteract)		n=536		Total	-4.6	60.8	-4.9	198	318	-3067	0.51	B2
Bent Beam at Shaft	S1	TBNT 553	92.9	Gravity	-10.3	-24.1	-0.2	0	4	-983		
				H. Seismic	22.5	-73.1	-8.2	-164	184	-1504		
				0.314 Gravity	3.2	-7.6	-0.1	0	1	-309		
(counteract)		n=508		Total	15.4	-104.7	-8.5	-164	190	-2796	1.56	B3
Bent conn at center of tower roof	S1/2,3	TBNT 531	-	Gravity	-7.0	0.6	0.1	-3	0	0		
				H. Seismic	-14.6	11.4	2.7	-134	0	0		
				0.314 Gravity	-2.2	0.2	0.0	-1	0	0		
(additive)		n=531		Total	-23.7	12.1	2.8	-138	0	0	0.33	B4
Bent/roof beam conn	S2	EB 509	-	Gravity	-4.2	9.7	0.0	1	0	0		
	S4/13,19			H. Seismic	-20.2	9.9	0.2	2	0	0		
	S5/17,18			0.314 Gravity	-1.3	3.0	0.0	0	0	0		
(additive)		n=514		Total	-25.6	22.7	0.1	3	0	0	0.26	B5
Corner mullions at their base	S4/A,2,3	CLCM 1	37.9	Gravity	-11.4	3.9	4.4	0	143	198		
				H. Seismic	-6.6	22.3	12.7	20	616	2461		
				0.314 Gravity	-3.6	1.2	1.4	0	45	62		
(additive)	col. 1-4	n=503		Total	-21.6	27.4	18.5	20	805	2722	1.47	B6a
Corner mullion/ 10WF conn	S4/A,2,5	CLCM 1	-	Gravity	-11.4	3.9	4.4	0	0	0		
				H. Seismic	6.6	22.3	12.7	20	0	0		
				0.314 Gravity	3.6	1.2	1.4	0	0	0		
(counteract)	col. 1-4	n=503		Total	-1.2	27.4	18.5	20	0	0	0.52	B7
Corner mullion/ 10WF conn	S4/8,9	CLCM 3	-	Gravity	-10.0	5.2	-4.1	0	0	0		
				H. Seismic	6.4	13.4	-9.8	104	0	0		
				0.314 Gravity	3.1	1.6	-1.3	0	0	0		
(counteract)	col. 5	n=506		Total	-0.4	20.2	-15.1	105	0	0	0.29	B8
10WF72 at 16WF88 shaft roof	S2,S3	RAD 525	43.4	Gravity	-0.2	9.4	0.0	0	0	155		
	S4/A			H. Seismic	-3.3	11.3	1.2	-1	12	426		
				0.314 Gravity	-0.1	2.9	0.0	0	0	49		
(additive)		n=515		Total	-3.5	23.6	1.2	-1	12	630	0.22	B9
Cab floor 12WF27 near Col 5	S2	CEPG 602	63.0	Gravity	5.6	-2.7	0.0	0	0	191		
				H. Seismic	8.1	-1.4	0.2	0	11	89		
				0.314 Gravity	1.8	-0.9	0.0	0	0	60		
(additive)		n=604		Total	15.5	-5.0	0.2	0	12	340	0.43	B10
C 4 x 7.25 at base of window	S3	CEPG 701	192.0	Gravity	0.0	0.0	-0.1	0	0	-4		
				H. Seismic	-4.5	0.7	0.0	0	0	-68		
				0.314 Gravity	0.0	0.0	0.0	0	0	-1		
(additive)		n=702		Total	-4.5	0.7	-0.1	0	0	-74	0.91	B11
Interior mullion (S3x7.5)	A5/5	WM 2	90.4	Gravity	0.3	0.0	0	0	0	0		
				H. Seismic	-0.1	0.0	0	0	0	-1		
				0.314 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	0.1	0.1	0	0	0	-2	0.03	B12
Corner mullion at the roof	S4/1,2,3,7	CUCM 7	26.7	Gravity	-5.2	0.8	0.0	0	1	21		
				H. Seismic	-4.3	3.0	2.6	-26	71	79		
				0.314 Gravity	-1.6	0.2	0.0	0	0	7		
(additive)		n=805		Total	-11.1	4.0	2.6	-27	71	108	0.11	B13
C 6 x 8.2 at top of window	S3	CEPG 808	237.0	Gravity	-0.1	0.0	0.0	0	0	4		
				H. Seismic	-9.4	1.2	0.2	0	0	95		
				0.314 Gravity	0.0	0.0	0.0	0	0	1		
(buckling)		n=808		Total	-9.5	1.2	0.2	0	0	101	0.76	B14
TS 20x4x1/2 around mull. perimeter	NA	TUBE 552	23.8	Gravity	-5.0	13.8	0.0	21	0	329		
				H. Seismic	-8.0	83.6	2.8	97	68	1990		
				0.314 Gravity	-1.6	4.3	0.0	7	0	103		
(additive)	retrofit member	n=506		Total	-14.6	101.7	2.8	125	68	2423	0.61	B15

6 San Luis Obispo Analysis Results

As with the Type L towers, the San Luis Obispo tower was analyzed using SAP2000 and MathCAD member and connection evaluation files. The first section of these results focuses on the tower shaft evaluation, which was controlled by the conservative assumption that all the cab windows remain intact and perform as fully effective shear walls (Cases SLO1, SLO2, SLO3, and SLO3a). The second section focuses on the cab evaluation, which was a more realistic model that does not include any stiffening from the cab windows. Table 7 summarized all the cases used to evaluate the San Luis Obispo tower.

Each section in this report presents the modal analysis results, deflections, story drifts, forces, and moments of critical members and connections and the resulting DCRs. The towers may fail evaluation based on either exceeding drift limits, or high DCRs that lead to collapse of the towers. Code-based design resistance factors have been dropped (i.e., set equal to 1.0) for the purpose of evaluating structural members. The design resistance factors were included in the evaluation of connections, as such failure must be prevented, because they would fail in a more brittle manner than the structural members would. Appendix D1 through D12 shows the MathCAD models used to evaluate each member and connection, and these appendix numbers are given in each force, moment, and DCR summary table in this chapter.

Shaft Members and Connections

The most critical condition for the shaft evaluation is the extreme condition where all cab windows remain intact and act as fully effective shear walls (SLO1 – SLO3a). This decreases the fundamental period of the structure and increases the effective acceleration, and therefore loads the shaft members and connections more severely. As with the Type L evaluation, this assumption provides an upper bound basis for evaluating the effect of the windows remaining intact. This effect was modeled by defining the shear stiffness of the windows if they act as a shear block. For the San Luis Obispo cab, the connections between the mullions and structural tubing above and below the windows are fixed, so no modification of these connections is needed to model the windows acting as shear walls (such a modification was needed for the Type L evaluation). Analysis

Cases SLO1, SLO2, and SLO3 evaluate the performance of the San Luis Obispo tower with the windows acting as shear walls with the full seismic response spectrum acting at 90, 45, and 0 degrees to the X-axis ($\theta_L = 90^\circ$, 45° , and 0°), respectively. The X-axis orientation is shown on the model plot in Figure 7. This axis is along the front face of the tower as shown on Drawing S-2 (Leo A Daly 1981) of the San Luis Obispo tower.

Modal Analysis Results and Deflections

Table 35 presents the primary modes of vibration for the San Luis Obispo tower, Cases SLO1, SLO2, and SLO3 evaluations. The cumulative participating mass shows that a greater portion of the mass participates in the first X and Y lateral modes than in the cases when the cab windows are not acting as shear walls (SLO5, SLO6, and SLO7 in Table 44). This demonstrates that the cab is more coupled with the tower shaft vibration in the first mode, and this will more heavily load the shaft components due to higher effective accelerations.

Table 36 summarizes maximum lateral deflections at each floor level and other key locations in the control tower for the shaft evaluation cases. All deflections are the SRSS of the total X and Y lateral deflections. The shaft deflections are greater for the shear wall window cases (SLO1 – SLO3) because of the coupling and higher effective accelerations described above. The greatest deflection at the top of the shaft is 1.32 in. for case SLO3 ($\theta_L = 0^\circ$). These deflections are well within story drift limits. Therefore, story drifts are not considered further in this section, but will be examined in the next section for the cab. Cab deflections for the cases of shear wall windows are unrealistically low because of the shear wall window stiffening. These cases are intended for shaft evaluation only, and will produce unrealistic results in the cab.

Shaft Member and Connection Evaluation

Tables 37 through 39 give the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. Tables 37 through 43 and 47 through 51 presents the same information for the San Luis Obispo tower evaluation as described in the Type L tower evaluation. For this tower, component capacities are defined in Appendices D1 through D12, using the forces and moments calculated from analysis case SLO1.

Case SLO3, with the full seismic spectrum at 0 degrees to the X-axis ($\theta_L = 0^\circ$), is the most critical loading condition for shaft members and connections. The highest DCR is in the braces. Table 39 shows that a brace in the plane of the X-

axis (BR107), located at the first floor of the building, has a DCR of 3.06. Other braces (such as BR508) shown in Table 39 have DCRs over 2.0. These high DCRs indicate that several braces will buckle. Tables 40 and 41 give the same information for all the braces in the plane of the X-axis. DCRs are much smaller for the braces out-of-plane to the X-axis, as the building is not significantly seismically loaded in this direction. Table 40 reveals that a brace at the third floor (BR407) has the highest DCR, with a value of 4.25. Table 40 also shows that all the braces up to the junction level (fourth floor) will buckle, as indicated by DCRs greater than 2.0.

The impact of buckling braces was studied further by removing all the braces on one side of the building, in the plane of the X-axis. In an earthquake, all the braces acting in compression on one side of the building could buckle, which would increase the load of the braces in tension on the other side. When the building cycles back in the other direction, the braces that had been in tension will go into compression and will also buckle, while the braces that had buckled on the other side will effectively carry load in tension. Therefore, if the braces in tension have adequate capacity alone for the full lateral load, the building will remain stable and prevent collapse. Many buildings are designed with tension only braces, where the design assumption is that the slender tension braces will buckle. The braces in the San Luis Obispo tower could have been designed as tension only, but this remains unclear. These braces are L6 x 6 x ½ with a kl/r value of only 124, which is not that slender.

The San Luis Obispo tower was evaluated with tension only braces in analysis Case SLO3a. The loading was identical to Case SLO3. Table 42 summarizes the forces, moments, and DCRs for all the tension braces in the plane of the X-axis. These braces are now loaded with greater axial tension force than when the compression braces were in place, and the DCRs are still much lower than the values of the compression braces in Case SLO3. The DCRs shown in Table 42 include the moments on these eccentrically loaded braces. Brace 202, between floors 2 and 3 has a high DCR, with a value of 2.09. This indicates that this brace will begin to yield in tension on one edge of the angle. However, the tower will remain very stable, because collapse would require yielding of the entire gross area of the section. This same brace was evaluated for gross section yielding with the same loading, which gives a DCR of only 0.53 (see Table 42). Therefore, the tower would remain stable, with only slight yielding of the braces in combined tension and moment. Deflections would be somewhat greater for the case with tension only braces than those shown in Table 36, for Case SLO3. However, they will remain far below acceptable story drift limits.

All the other components were evaluated with the tension only analysis case (SLO3a), and a summary of this evaluation is shown in Table 43. Comparing Table 43 with Table 39 (Case SLO3) shows that the loading and DCRs for many components increase with the tension only braces. Most significant are the increases for the column at the base (0.83 to 1.70 DCR increase); beam at an intermediate level (0.52 to 1.90); brace connection at the tower base (0.66 to 1.22); beam at junction level (0.46 to 1.07); and beams at the base of the mullions (0.51 to 1.05). These values still fall below 2.0. Therefore, the seismic performance of the San Luis Obispo tower shaft is adequate and needs no upgrade.

Table 35. San Luis Obispo Shaft Evaluation Modal Analysis Results.

Mode #	SLO1, SLO2 & SLO3				Mode of Vibration
	Period (sec)	Cumulative Participating Mass (%)			
		X-dir	Y-dir	Z-dir	
1	0.390	3.9	48.8	0.0	1 st Y-Lateral
2	0.382	64.8	51.9	0.0	1 st X-Lateral
3	0.266	65.0	51.9	0.0	1 st Torsion
4	0.193	83.3	52.8	0.0	2 nd X-Lateral
5	0.184	84.0	81.1	0.0	2 nd Y-Lateral
6	0.142	84.6	81.6	0.0	2 nd Torsion
7	0.135	84.6	81.6	7.4	1 st Vertical – 4 th floor/junction
8	0.127	85.4	82.2	9.2	2 nd Vertical – 2 nd floor
9	0.126	85.4	82.5	9.5	3 rd Vertical – 2 nd & 3 rd floor
10	0.125	85.5	82.7	15.0	4 th Vertical – 3 rd floor
11	0.123	85.6	82.7	17.4	5 th Vertical – 4 th floor, cab roof/floor
12	0.116	86.4	82.7	21.3	6 th Vertical – 2 nd floor
15	0.112	86.6	84.4	28.1	7 th Vertical –cab roof
16	0.108	92.7	85.7	28.1	3 rd X-Lateral
17	0.100	94.0	89.7	28.2	3 rd Y-Lateral
19	0.094	94.3	89.9	41.7	7 th Vertical – cab floor & roof
24	0.083	95.1	90.3	51.1	9 th Vertical – cab rocking

Table 36. San Luis Obispo Shaft Evaluation Selected Horizontal SRSS Deflections.

Location	SLO1		SLO2		SLO3	
	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	217	0.16	217	0.17	217	0.20
3 rd Floor, δ_2	316	0.36	316	0.40	316	0.45
4 th Floor/ Junction, δ_3	520	0.59	520	0.63	526	0.72
Top of Shaft, δ_4	824	1.13	824	1.20	824	1.32
Cab Floor, δ_5	920	1.76	920	1.81	920	1.82
Bottom of Windows	1008	2.14	1008	2.17	1012	2.15
Top of Windows	1108	2.61	1108	2.65	1112	2.63
Cab Roof, δ_6	1220	2.79	1220	2.84	1227	2.82
Parapet	1308	3.06	1308	3.12	1312	3.10

Table 37. Forces, Moments, and DCRs (SLO1, all windows, $\theta_L = 90^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 104	149.1	Gravity	-3.2	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-47.3	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=101		0.300 Gravity	-1.0	0.0	0.0	0	0	0		
				Total	-51.5	-0.1	0.0	0	0	0	1.89	D1
Column at Base	S-4	C 101	118.5	Gravity	-45.6	0.0	0.0	0	0	-1		
				H. Seismic	-116.0	-0.7	-0.2	0	-16	-84		
(additive)	W8x31	n=101		0.300 Gravity	-13.7	0.0	0.0	0	0	0		
				Total	-175.3	-0.8	-0.2	0	-16	-85	0.70	D2
Beam at Intermediate Levels	S-2, S-4 W14x34 W16x40	I 210	22.6	Gravity	0.0	1.8	0.0	0	0	-69		
(additive)	W16x45	n=210		H. Seismic	0.0	8.4	0.0	0	0	-759		
				0.300 Gravity	0.0	0.5	0.0	0	0	-21		
				Total	0.0	10.7	0.0	0	0	-849	0.33	D3
Bracing conn at base	S-4	BR 104	-	Gravity	-3.2	-0.1	0.0	0	0	0		
- Tension				H. Seismic	47.3	0.0	0.0	0	0	0		
(counteract)		n=101		0.300 Gravity	1.0	0.0	0.0	0	0	0		
				Total	45.0	-0.1	0.0	0	0	0	0.57	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4 W14x34 W16x40	J 512	90.5	Gravity	0.0	-7.0	0.0	0	0	-34		
(additive)	W16x45	n=511		H. Seismic	0.0	-7.1	0.0	0	0	-781		
				0.300 Gravity	0.0	-2.1	0.0	0	0	-10		
				Total	0.0	-16.2	0.0	0	0	-825	0.32	D5
Bracing btwn Junct & Wlkwy	S-4	BR 503	102.3	Gravity	-4.8	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-52.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=503		0.300 Gravity	-1.4	0.0	0.0	0	0	0		
				Total	-58.4	0.0	0.0	0	0	0	1.57	D6
Beams at Base of mullions	S-2, S-3 W8x35, W10x26	TS 807	82.6	Gravity	0.6	0.9	0.0	0	-2	117		
(additive)	W16x77	n=808		H. Seismic	8.5	6.2	-1.9	1	-152	511		
				0.300 Gravity	0.2	0.3	0.0	0	-1	35		
				Total	9.2	7.4	-1.9	1	-154	663	0.81	D7
Corner mullion connections at base	S-3	MUL 804	40.8	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
(counteract)		n=824		H. Seismic	9.5	-1.8	-13.5	-49	-454	-110		
				0.300 Gravity	1.9	0.0	-0.2	0	-7	-3		
				Total	5.1	-1.9	-14.3	-49	-484	-121	1.93	D8
Mullions at top & btm of window	S-5	MUL 804	40.8	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
(additive)	TS			H. Seismic	-9.5	-1.8	-13.5	-49	-454	-110		
	8x4x1/2	n=824		0.300 Gravity	-1.9	0.0	-0.2	0	-7	-3		
				Total	-17.5	-1.9	-14.3	-49	-484	-121	0.84	D9
Mullions within window span	S-5	MUL 1002	94.8	Gravity	-2.8	-0.3	-0.1	0	-8	-13		
(additive)	TS			H. Seismic	-5.8	-3.4	-5.8	1	-296	-300		
	8x4x1/2	n=1004		0.300 Gravity	-0.8	-0.1	0.0	0	-2	-4		
				Total	-9.5	-3.8	-5.9	1	-307	-316	0.78	D10
Tubes at base of window	S-5	BW 1007	57.5	Gravity	-0.3	-0.2	-0.1	0	3	11		
(additive)	TS			H. Seismic	-9.6	-13.4	-2.3	6	141	377		
	7x7 3/16	n=1006		0.300 Gravity	-0.1	-0.1	0.0	0	1	3		
				Total	-10.0	-13.7	-2.4	6	145	390	1.10	D11
Tubes at top of window	S-5	TW 1107	72.4	Gravity	0.3	0.0	0.0	0	0	6		
(additive)	TS			H. Seismic	6.9	-6.6	1.9	1	-73	237		
	7x7 3/16	n=1106		0.300 Gravity	0.1	0.0	0.0	0	0	2		
				Total	7.3	-6.6	1.9	1	-74	244	0.62	D12

Table 38. Forces, Moments, and DCRs (SLO2, all windows, $\theta_L = 45^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 108	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-40.6	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		0.300 Gravity	-2.2	0.0	0.0	0	0	0		
				Total	-50.2	-0.1	0.0	0	0	0	1.81	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-99.5	-0.6	-0.3	0	-33	-73		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-168.8	-0.7	-0.3	0	-33	-74	0.70	D2
Beam at Intermediate Levels	S-2, S-4 W14x34	I 223	50.0	Gravity	0.0	0.2	0.0	0	0	-40		
	W16x40			H. Seismic	0.0	25.3	0.0	0	0	-654		
(additive)	W16x45	n=220		0.300 Gravity	0.0	0.1	0.0	0	0	-12		
				Total	0.0	25.6	0.0	0	0	-707	0.37	D3
Bracing conn at base	S-4	BR 108	-	Gravity	-7.4	-0.1	0.0	0	0	0		
- Tension				H. Seismic	40.6	0.0	0.0	0	0	0		
(counteract)		n=105		-0.300 Gravity	2.2	0.0	0.0	0	0	0		
				Total	35.3	-0.1	0.0	0	0	0	0.44	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4 W14x34	J 512	90.5	Gravity	0.0	-7.0	0.0	0	0	-34		
(additive)	W16x40			H. Seismic	0.0	-8.5	0.0	0	0	-702		
	W16x45	n=511		0.300 Gravity	0.0	-2.1	0.0	0	0	-10		
				Total	0.0	-17.7	0.0	0	0	-746	0.29	D5
Bracing btwn Junct & Wlkwy	S-4	BR 508	120.0	Gravity	-9.4	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-48.1	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		0.300 Gravity	-2.8	0.0	0.0	0	0	0		
				Total	-60.3	-0.1	0.0	0	0	0	1.84	D6
Beams at Base of mullions	S-2, S-3 W8x35,	TS 807	82.04	Gravity	0.6	0.9	0.0	0	-2	117		
(additive)	W10x26			H. Seismic	10.7	5.7	-1.7	1	-140	467		
	W16x77	n=808		0.300 Gravity	0.2	0.3	0.0	0	-1	35		
				Total	11.4	6.8	-1.7	1	-143	619	0.76	D7
Corner mullion connections at base	S-3	MUL 804	40.75	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
(counteract)		n=824		H. Seismic	14.2	-3.0	-12.4	-45	-428	-105		
				0.300 Gravity	1.9	0.0	-0.2	0	-7	-3		
				Total	9.8	-3.1	-13.2	-45	-458	-116	1.86	D8
Mullions at top & btm of window	S-5	MUL 906	37.5	Gravity	-7.0	0.4	0.0	0	-1	0		
(additive)	TS			H. Seismic	-21.7	13.1	3.0	-8	-189	-630		
	8x4x1/2	n=1012		0.300 Gravity	-2.1	0.1	0.0	0	0	0		
				Total	-30.7	13.6	3.1	-8	-190	-631	0.88	D9
Mullions within window span	S-5	MUL 1006	94.63	Gravity	-5.0	-0.5	-0.1	0	-3	-25		
(additive)	TS			H. Seismic	-9.4	-4.5	-4.4	2	-229	-414		
	8x4x1/2	n=1012		0.300 Gravity	-1.5	-0.1	0.0	0	-1	-7		
				Total	-16.0	-5.1	-4.5	2	-234	-446	0.83	D10
Tubes at base of window	S-5	BW 1005	57.5	Gravity	-0.2	0.5	-0.1	0	6	-8		
(additive)	TS			H. Seismic	-8.6	12.0	-2.5	5	125	-352		
	7x7 3/16	n=1004		0.300 Gravity	-0.1	0.1	0.0	0	2	-2		
				Total	-8.9	12.6	-2.6	5	133	-363	1.01	D11
Tubes at top of window	S-5	TW 1109	72.4	Gravity	0.5	-0.2	-0.1	0	-2	-4		
(additive)	TS			H. Seismic	8.9	-6.0	-1.4	1	-66	-215		
	7x7 3/16	n=1110		0.300 Gravity	0.2	-0.1	0.0	0	-1	-1		
				Total	9.6	-6.3	-1.5	1	-68	-220	0.56	D12

Table 39. Forces, Moments, and DCRs (SLO3, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 107	149.1	Gravity	-7.6	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-57.9	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=104		0.300 Gravity	-2.3	0.0	0.0	0	0	0		
				Total	-67.8	-0.1	0.0	0	0	0	3.06	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-136.7	-0.3	-0.4	0	-45	-37		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-206.0	-0.4	-0.4	0	-45	-38	0.83	D2
Beam at Intermediate Levels	S-2, S-4 W14x34	I 223	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
				H. Seismic	0.0	-36.2	0.0	0	0	-921		
(additive)	W16x40	n=219		0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
	W16x45			Total	0.0	-37.3	0.0	0	0	-994	0.52	D3
Bracing conn at base	S-4	BR 107	-	Gravity	-7.6	0.1	0.0	0	0	0		
- Tension				H. Seismic	57.9	0.0	0.0	0	0	0		
(counteract)		n=104		-0.300 Gravity	2.3	0.0	0.0	0	0	0		
				Total	52.6	0.1	0.0	0	0	0	0.66	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4 W14x34	J 527	50.0	Gravity	0.0	-2.1	0.0	0	0	-78		
				H. Seismic	0.0	-29.8	0.0	0	0	-769		
(additive)	W16x40	n=523		0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
	W16x45			Total	0.0	-32.5	0.0	0	0	-870	0.46	D5
Bracing btwn Junct & Wlkwy	S-4	BR 508	120.0	Gravity	-9.4	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-70.1	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		0.300 Gravity	-2.8	0.0	0.0	0	0	0		
				Total	-82.2	-0.1	0.0	0	0	0	2.91	D6
Beams at Base of mullions	S-2, S-3 W8x35, W10x26	TS 833	43.6	Gravity	-3.6	-7.7	-0.2	0	-12	-68		
				H. Seismic	-8.3	-48.8	-7.3	-6	-287	-1486		
(additive)	W16x77	n=827		0.300 Gravity	-1.1	-2.3	-0.1	0	-3	-20		
				Total	-12.9	-58.9	-7.6	-6	-302	-1575	0.51	D7
Corner mullion connections at base (counteract)	S-3	MUL 805	40.8	Gravity	-6.0	-0.1	0.7	-2	24	-6		
				H. Seismic	16.8	-2.2	14.9	-53	488	-131		
				0.300 Gravity	1.8	0.0	0.2	0	7	-2		
		n=825		Total	12.6	-2.3	15.8	-55	519	-139	2.14	D8
Mullions at top & btm of window	S-5	MUL 905	37.5	Gravity	-3.3	0.1	-0.2	0	3	-4		
				H. Seismic	-16.5	13.6	-3.1	-10	228	-455		
(additive)	TS	n=1009		0.300 Gravity	-1.0	0.0	-0.1	0	1	-1		
	8x4x1/2			Total	-20.9	13.7	-3.3	-10	232	-460	0.77	D9
Mullions within window span	S-5	MUL 1006	94.6	Gravity	-5.0	-0.5	-0.1	0	-3	-25		
				H. Seismic	-5.0	-2.5	-6.3	1	-328	-235		
(additive)	TS	n=1012		0.300 Gravity	-1.5	-0.1	0.0	0	-1	-7		
	8x4x1/2			Total	-11.6	-3.1	-6.4	1	-332	-267	0.79	D10
Tubes at base of window	S-5	BW 1012	57.6	Gravity	-0.9	0.5	0.2	0	5	21		
				H. Seismic	-9.5	13.2	1.2	-6	120	373		
(additive)	TS	n=1011		0.300 Gravity	-0.3	0.2	0.1	0	1	6		
	7x7 3/16			Total	-10.7	13.9	1.4	-7	126	400	1.09	D11
Tubes at top of window	S-5	TW 1110	72.4	Gravity	0.4	0.3	0.1	0	-3	-6		
				H. Seismic	10.5	6.5	1.5	-1	-67	-234		
(additive)	TS	n=1111		0.300 Gravity	0.1	0.1	0.0	0	-1	-2		
	7x7 3/16			Total	11.0	6.9	1.6	-1	-70	-242	0.61	D12

Table 40. Braces Between Foundation and Junction Level (SLO3, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn Fndtn & I2 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 101 n=101	149.1	Gravity H. Seismic 0.300 Gravity Total	-6.4 -57.7 -1.9 -66.0	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.89	D1
Bracing btwn Fndtn & I2 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 102 n=102	149.1	Gravity H. Seismic 0.300 Gravity Total	-6.6 -58.4 -2.0 -66.9	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.97	D1
Bracing btwn Fndtn & I2 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 107 n=104	149.1	Gravity H. Seismic 0.300 Gravity Total	-7.6 -57.9 -2.3 -67.8	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	3.06	D1
Bracing btwn Fndtn & I2 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 108 n=105	149.1	Gravity H. Seismic 0.300 Gravity Total	-7.5 -57.4 -2.2 -67.1	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.99	D1
Bracing btwn I2 & I3 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 201 n=201	150.3	Gravity H. Seismic 0.300 Gravity Total	-6.3 -54.4 -1.9 -62.6	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.61	D1
Bracing btwn I2 & I3 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 202 n=205	150.3	Gravity H. Seismic 0.300 Gravity Total	-6.2 -54.7 -1.9 -62.7	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.62	D1
Bracing btwn I2 & I3 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 207 n=217	150.3	Gravity H. Seismic 0.300 Gravity Total	-7.2 -58.5 -2.2 -67.9	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	3.07	D1
Bracing btwn I2 & I3 L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 208 n=221	150.3	Gravity H. Seismic 0.300 Gravity Total	-7.3 -58.1 -2.2 -67.6	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	3.04	D1
Bracing btwn I3 & JNCTN L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 401 n=301	150.3	Gravity H. Seismic 0.300 Gravity Total	-13.1 -45.7 -3.9 -62.7	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	2.62	D1
Bracing btwn I3 & JNCTN L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 402 n=305	150.3	Gravity H. Seismic 0.300 Gravity Total	-12.0 -53.8 -3.6 -69.4	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	3.22	D1
Bracing btwn I3 & JNCTN L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 407 n=316	150.3	Gravity H. Seismic 0.300 Gravity Total	-14.4 -58.9 -4.3 -77.6	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	4.25	D1
Bracing btwn I3 & JNCTN L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 408 n=320	150.3	Gravity H. Seismic 0.300 Gravity Total	-15.5 -48.1 -4.7 -68.3	0.0 0.0 0.0	0.0 0.0 0.0	0 0 0	0 0 0	0 0 0	3.11	D1

Table 41. Braces Between Junction and Top of Shaft (SLO3, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn JNCTN&CBACC	S-4	BR 501	103.0	Gravity	-7.3							
L6x6x1/2				H. Seismic	-46.9							
(Buckling)	L6x6x1/2	n=501		0.300 Gravity	-2.2	0.0	0.0	0	0	0		
				Total	-56.5	0.0	0.0	0	0	0	1.51	D1
Bracing btwn JNCTN&CBACC	S-4	BR 502	101.4	Gravity	-8.1							
L6x6x1/2				H. Seismic	-48.1							
(Buckling)	L6x6x1/2	n=506		0.300 Gravity	-2.4	0.0	0.0	0	0	0		
				Total	-58.6	0.0	0.0	0	0	0	1.58	D1
Bracing btwn JNCTN&CBACC	S-4	BR 507	98.7	Gravity	-10.9							
L6x6x1/2				H. Seismic	-66.0							
(Buckling)	L6x6x1/2	n=520		0.300 Gravity	-3.3	0.0	0.0	0	0	0		
				Total	-80.1	0.0	0.0	0	0	0	2.35	D1
Bracing btwn JNCTN&CBACC	S-4	BR 508	120.05	Gravity	-9.4							
L6x6x1/2				H. Seismic	-70.1							
(Buckling)	L6x6x1/2	n=526		0.300 Gravity	-2.8	0.0	0.0	0	0	0		
				Total	-82.2	0.0	0.0	0	0	0	2.91	D1
Bracing btwn CBACC&TofS	S-4	BR 701	90.5	Gravity	-7.1							
L6x6x1/2				H. Seismic	-47.0							
(Buckling)	L6x6x1/2	n=602		0.300 Gravity	-2.1	0.0	0.0	0	0	0		
				Total	-56.2	0.0	0.0	0	0	0	1.40	D1
Bracing btwn CBACC&TofS	S-4	BR 702	89.1	Gravity	-7.9							
L6x6x1/2				H. Seismic	-48.0							
(Buckling)	L6x6x1/2	n=605		0.300 Gravity	-2.4	0.0	0.0	0	0	0		
				Total	-58.2	0.0	0.0	0	0	0	1.46	D1
Bracing btwn CBACC&TofS	S-4	BR 707	86.8	Gravity	-10.6							
L6x6x1/2				H. Seismic	-65.9							
(Buckling)	L6x6x1/2	n=614		0.300 Gravity	-3.2	0.0	0.0	0	0	0		
				Total	-79.7	0.0	0.0	0	0	0	2.06	D1
Bracing btwn CBACC&TofS	S-4	BR 708	70.5	Gravity	-8.6							
L6x6x1/2				H. Seismic	-59.1							
(Buckling)	L6x6x1/2	n=710		0.300 Gravity	-2.6	0.0	0.0	0	0	0	*	
				Total	-70.3	0.0	0.0	0	0	0	0.12	D1

* Low DCR because the brace length is much less.

Table 42. Tension Braces Only (SLO3a, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn Fndtn & I2 L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 102 n=102	149.1	Gravity H. Seismic 0.300 Gravity Total	0.8 95.9 0.2 97.0	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.85	D1a
Bracing btwn Fndtn & I2 L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 108 n=105	149.1	Gravity H. Seismic -0.300 Gravity Total	-5.0 66.6 1.5 63.1	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.21	D1a
Bracing btwn I2 & I3 L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 202 n=205	150.3	Gravity H. Seismic 0.300 Gravity Total	3.3 105.1 1.0 109.4	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	2.09 (w/bending) 0.53 (wo/bending)	D1a
Bracing btwn I2 & I3 L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 208 n=221	150.3	Gravity H. Seismic -0.300 Gravity Total	-2.8 75.2 0.8 73.3	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.40	D1a
Bracing btwn I3 & JNCTN L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 402 n=305	150.3	Gravity H. Seismic 0.300 Gravity Total	4.0 88.6 1.2 93.7	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.79	D1a
Bracing btwn I3 & JNCTN L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 408 n=320	150.3	Gravity H. Seismic -0.300 Gravity Total	-3.7 77.3 1.1 74.7	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.43	D1a
Bracing btwn JNCTN&CBACC L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 502 n=506	101.4	Gravity H. Seismic -0.300 Gravity Total	-0.6 68.9 0.2 68.4	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.31	D1a
Bracing btwn JNCTN&CBACC L6x6x1/2 (Tension)	S-4 L6x6x1/2	BR 508 n=526	120.05	Gravity H. Seismic -0.300 Gravity Total	-1.0 84.0 0.3 83.3	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.59	D1a
Bracing btwn CBACC&TofS L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 702 n=605	89.1	Gravity H. Seismic -0.300 Gravity Total	-0.4 68.9 0.1 68.6	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.31	D1a
Bracing btwn CBACC&TofS L6x6x1/2 (Buckling)	S-4 L6x6x1/2	BR 708 n=710	70.5	Gravity H. Seismic -0.300 Gravity Total	-0.5 76.1 0.2 75.7	 0.0 0.0	 0.0 0.0	 0 0	 0 0	 0 0	1.45	D1a

Table 43. Other Components w/Tension Braces Only (SLO3a, all windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 102	-	Gravity	0.8	0.0	0.0	0	0	0		
- Tension				H. Seismic	95.9	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=102		0.300 Gravity	0.2	0.0	0.0	0	0	0		
				Total	97.0	0.0	0.0	0	0	0	1.85	D1a
Column at Base	S-4	C 104	118.5	Gravity	-53.4	0.0	0.6	0	59	-3		
				H. Seismic	-98.4	-0.3	4.3	0	459	-34		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.2	0	18	-1		
				Total	-167.9	-0.3	5.1	0	536	-38	1.70	D2
Beam at Intermediate Levels	S-2, S-4 W14x34 W16x40	I 203	50.0	Gravity	0.0	0.0	0.0	0	0	440		
(additive)	W16x45	n=204		H. Seismic	0.0	29.9	0.0	0	0	4261		
				0.300 Gravity	0.0	0.0	0.0	0	0	138		
				Total	0.0	29.9	0.0	0	0	4840	1.90	D3
Bracing conn at base	S-4	BR 102	-	Gravity	0.6	-0.1	0.0	0	0	0		
- Tension				H. Seismic	95.9	0.0	0.0	0	0	0		
(additive)		n=102		0.300 Gravity	0.2	0.0	0.0	0	0	0		
				Total	96.7	-0.1	0.0	0	0	0	1.22	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4 W14x34 W16x40	J 503	102.3	Gravity	0.0	-1.3	0.0	0	0	448		
(additive)	W16x45	n=504		H. Seismic	0.0	-24.6	0.0	0	0	2146		
				0.300 Gravity	0.0	-0.4	0.0	0	0	134		
				Total	0.0	-26.3	0.0	0	0	2729	1.07	D5
Bracing btwn Junct & Wlkwy	S-4	BR 202	-	Gravity	3.3	0.0	0.0	0	0	0		
- Tension				H. Seismic	105.1	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=205		0.300 Gravity	1.0	0.0	0.0	0	0	0		
				Total	109.4	0.0	0.0	0	0	0	2.09	D1a
Beams at Base of mullions	S-2, S-3 W8x35, W10x26	TS 834	75.1	Gravity	0.9	1.3	0.6	1	-19	864		
(additive)	W16x77	n=829		H. Seismic	27.1	55.3	2.5	6	-59	4134		
				0.300 Gravity	0.3	0.4	0.2	0	-6	259		
				Total	28.2	57.0	3.2	7	-83	5257	1.05	D7
Corner mullion connections at base (counteract)	S-3	MUL 806	40.75	Gravity	-1.9	-1.3	0.0	0	-1	51		
				H. Seismic	38.7	-4.4	11.9	17	-97	168		
(additive)		n=828		0.300 Gravity	0.6	-0.4	0.0	0	0	15		
				Total	37.4	-6.1	11.9	17	-98	235	1.07	D8
Mullions at top & btm of window	S-5	MUL 902	37.5	Gravity	-8.2	-0.3	0.2	-1	-10	0		
(additive)	TS 8x4x1/2	n=1004		H. Seismic	-26.3	-6.1	4.4	-6	-159	290		
				0.300 Gravity	-2.5	-0.1	0.1	0	-3	0		
				Total	-37.0	-6.4	4.7	-6	-172	290	0.56	D9
Mullions within window span	S-5	MUL 1005	94.8	Gravity	-4.6	-0.7	1.1	0	69	-46		
(additive)	TS 8x4x1/2	n=1009		H. Seismic	-4.6	-4.0	3.9	1	249	-337		
				0.300 Gravity	-1.4	-0.2	0.3	0	21	-14		
				Total	-10.6	-4.9	5.4	1	339	-397	0.91	D10
Tubes at base of window	S-5	BW 1011	57.6	Gravity	2.2	2.8	0.6	-1	-36	-74		
(additive)	TS 7x7 3/16	n=1010		H. Seismic	12.7	17.4	4.4	-3	-182	-495		
				0.300 Gravity	0.7	0.8	0.2	0	-11	-22		
				Total	15.6	21.0	5.2	-4	-229	-591	1.57	D11
Tubes at top of window	S-5	TW 1111	72.4	Gravity	-1.8	1.5	0.0	0	-25	-50		
(additive)	TS 7x7 3/16	n=1110		H. Seismic	-10.9	9.1	0.8	1	-131	-323		
				0.300 Gravity	-0.5	0.5	0.0	0	-7	-15		
				Total	-13.2	11.0	0.8	1	-163	-388	1.23	D12

Cab Members and Connections

The cab members and connections of the San Luis Obispo tower were evaluated with cases SLO4, SLO5, SLO6, and SLO7. Case SLO4 is for one window acting as a shear wall and cases SLO5 through SLO7 represent the most realistic condition where no windows act as a shear wall. The following sections summarize the San Luis Obispo cab evaluation in terms of modal behavior, deflections, story drifts, and component forces and moments used to calculate DCRs.

The SLO4 case assumes the window acting as a shear wall is located farthest from the center of mass (for 5 percent accidental eccentricity), which is placed farthest from the plane of this shear wall window. This will create the greatest distance between the center of mass and center of stiffness in the plan of the building and will create the greatest torsional response of the tower shaft. The resultant direction of the 100 percent full seismic spectrum loading plus the 30 percent orthogonal spectrum is parallel to this shear wall window, to create the worst torsional response.

Modal Analysis Results and Deflections

Table 44 presents the primary modes of vibration for the San Luis Obispo tower, evaluated in the SLO4, SLO5, SLO6, and SLO7 analysis cases. The cumulative participating mass shows that a smaller portion of the mass participates in the first X and Y lateral modes than in the cases where cab windows act as shear walls (SLO1, SLO2, and SLO3 in Table 35).

Table 45 presents the maximum lateral deflections at each floor level and other key locations in the San Luis Obispo control tower. Table 46 shows the greatest San Luis Obispo story drifts, which were for Case SLO7 ($\theta_L = 0^\circ$) for shaft and Case SLO5 ($\theta_L = 90^\circ$) for the cab. The calculated story drifts, Δ_c , fall below the allowable values, Δ_a , at each floor level. Therefore, no stiffening upgrade is needed to reduce deflections and story drifts.

Cab Member and Connection Evaluation

Tables 47 through 50 give the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. Case SLO7 (Table 50), with the full seismic spectrum at 0 degrees to the X-axis ($\theta_L = 0^\circ$), is the most critical loading condition for the cab members and connections. The highest DCR is for the cab column connections (window corner mullions). Table 50 shows that the most heavily loaded

Table 44. San Luis Obispo Cab Evaluation Modal Analysis Results.

Mode #	SLO5, SLO6, and SLO7				Mode of Vibration	SLO4 Period (sec)
	Period (sec)	Cumulative Participating Mass (%)				
		X-dir	Y-dir	Z-dir		
1	0.499	0.1	32.4	0.0	1 st Y-Lateral	0.483
2	0.465	40.3	32.4	0.0	1 st X-Lateral	0.441
3	0.338	40.3	32.4	0.0	1 st Torsion	0.299
4	0.255	80.7	32.7	0.0	2 nd X-Lateral	0.242
5	0.234	81.0	75.0	0.0	2 nd Y-Lateral	0.227
6	0.165	81.7	75.4	0.0	2 nd Torsion	0.150
7	0.135	81.8	75.4	7.4	1 st Vertical – 4 th floor/junction	0.135
8	0.132	82.1	77.9	7.4	3 rd Y-Lateral	
9	0.127	83.3	79.4	10.8	2 nd Vertical – 2 nd & 3 rd floor	0.127
10	0.126	83.3	86.4	11.1	3 rd Y-Lateral	0.123
11	0.125	83.3	86.7	13.0	3 rd Vertical – 2 nd & 3 rd floor	0.125
12	0.123	83.3	86.7	16.1	4 th Vertical – 4 th floor, cab roof/floor	0.123
13	0.119	89.0	86.7	20.6	3 rd X-Lateral & Vertical – 2 nd floor	0.118
14	0.115	89.1	86.7	20.8	5 th Vertical – 2 nd and 3 rd floor	0.115
15	0.113	91.7	86.8	23.4	3 rd X-Lateral – cab rocking	0.113
16	0.112	91.7	86.8	28.0	5 th Vertical –cab roof	0.112
20	0.094	93.4	88.5	33.7	6 th Vertical – cab floor	0.093
21	0.093	93.4	88.5	43.3	7 th Vertical – cab floor & roof	0.092
23	0.089	94.9	89.3	43.6	4 th X-Lateral	0.087
25	0.084	95.3	89.5	47.0	8 th Vertical – cab access stair support	
26	0.083	95.3	91.7	50.2	9 th Vertical/Y-Lateral – cab rocking	0.083

Table 45. San Luis Obispo Cab Evaluation Selected Horizontal SRSS Deflections.

Location	SLO4		SLO5		SLO6		SLO7	
	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)	Joint #	δ_{xe} (in.)
2 nd Floor, δ_1	217	0.18	217	0.13	217	0.16	217	0.18
3 rd Floor, δ_2	316	0.40	316	0.30	316	0.36	316	0.40
4 th Floor/ Junction, δ_3	520	0.62	520	0.49	520	0.56	520	0.62
Top of Shaft, δ_4	824	1.11	804	0.94	824	1.01	824	1.09
Cab Floor, δ_5	922	1.63	920	1.64	920	1.60	920	1.56
Bottom of Windows	1012	2.28	1008	2.22	1008	2.16	1012	2.10
Top of Windows	1112	4.28	1108	3.93	1108	3.87	1112	3.87
Cab Roof, δ_6	1227	4.70	1220	4.39	1220	4.34	1227	4.35
Parapet	1312	5.27	1308	5.02	1308	4.98	1312	5.00

of these connections has a DCR of 2.49. The TS 8 x 4 x ½ structural tubing mullion is welded to a 1-1/8-in.-thick base plate with a 5/16-in. fillet weld (San Luis Obispo tower Drawing No. S-5; Leo A Daly 1981). The critical mode of failure is shearing through the throat of the weld (see Appendix D8). The axial load in the tubing due to horizontal and vertical seismic load could be either positive (tensile) or negative, but it is assumed to be positive, which counteracts the

effects of gravity. This is because only tensile forces at this joint will load the weld in shear. Compressive force would simply transfer the load through bearing, not loading the critical weld. Only the connections of these mullions have DCRs greater than 2.0.

If one mullion connection begins to fail, loads would be redistributed to the other mullions because the mullions will all deflect together, as they are tied to each other at the cab floor. Therefore, the condition of all the mullion connections at their base must be examined and, if their average DCR exceeds 2.0, failure will begin. Table 51 gives the forces, moments, and DCRs for all the mullion connections to their base plates. The average DCR for all mullions is 1.97 as shown in the table. This is slightly less than the 2.0 value that defines failure. Weld failure would be brittle and must be prevented, so the design resistance factors are conservatively left in the evaluation (Appendix D8).

Channels that support the catwalk are connected (shear connections) to these mullions all the way around the cab perimeter. These channels are located just above the critical mullion base connections, but are not included in the San Luis Obispo model. The cab floor beams also frame into the mullions at a higher level with shear connections. Both the catwalk channels and cab floor beams will provide some moment resistance, reducing rotations and deflections at these levels and reducing the moment applied to the mullion base connections.

The design resistance factors remaining in the mullion connection evaluation and the catwalk channels and cab floor beam connections all reduce the stress condition of the critical mullion connections, and these effects were not accounted for in this evaluation. These unaccounted for contributions further ensure that the critical mullion connections (DCR = 1.97) will not fail in the maximum considered earthquake motions. Therefore, this tower has met the requirements of the life-safety evaluation and no upgrade is needed.

Table 46. San Luis Obispo Tower Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)
Ground Floor	-9.5			SLO7	104	0.00	
2nd Floor	109	118.5	2.37	SLO7	217	0.18	0.18
3rd Floor	229	120	2.4	SLO7	316	0.40	0.22
4th Floor	349	120	2.4	SLO7	520	0.62	0.22
Shaft Roof	520	171	3.42	SLO7	826	1.03	0.41
Shaft Roof	520	171	3.42	SLO5	824	0.92	
Cab Floor	560.8	40.75	0.815	SLO5	918	1.56	0.64
Cab Roof	718.8	158	3.16	SLO5	1220	4.39	2.83

Table 47. Forces, Moments, and DCRs (SLO4, one window, $\theta_L = 16.7^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 108	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-49.4	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		0.300 Gravity	-2.2	0.0	0.0	0	0	0		
				Total	-59.1	-0.1	0.0	0	0	0	2.36	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-81.1	-0.5	0.4	0	39	-51		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-150.3	-0.5	0.4	0	39	-52	0.63	D2
Beam at Intermediate Levels	S-2, S-4 W14x34	I 223	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
	W16x40			H. Seismic	0.0	-30.9	0.0	0	0	-767		
(additive)	W16x45	n=219		0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
				Total	0.0	-32.0	0.0	0	0	-840	0.44	D3
Bracing conn at base	S-4	BR 108	-	Gravity	-7.4	-0.1	0.0	0	0	0		
- Tension				H. Seismic	49.4	0.0	0.0	0	0	0		
(counteract)		n=105		-0.300 Gravity	2.2	0.0	0.0	0	0	0		
				Total	44.2	-0.1	0.0	0	0	0	0.56	D4
Beam at Jnct. level (El. 29'-8")	S-2, S-4 W14x34	J 527	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
	W16x40			H. Seismic	0.0	-25.3	0.0	0	0	-636		
(additive)	W16x45	n=523		0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
				Total	0.0	-28.0	0.0	0	0	-737	0.39	D5
Bracing btwn Jnct & Wlkwy	S-4	BR 508	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-62.8	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		0.300 Gravity	-2.7	0.0	0.0	0	0	0		
				Total	-74.6	-0.1	0.0	0	0	0	2.49	D6
Beams at Base of mullions	S-2, S-3 W8x35,	TS 833	43.6	Gravity	-3.4	-7.4	-0.2	0	-12	-65		
	W10x26			H. Seismic	-8.1	-42.4	-5.2	-4	-276	-1301		
(additive)	W16x77	n=827		0.300 Gravity	-1.0	-2.2	-0.1	0	-4	-20		
				Total	-12.6	-51.9	-5.5	-5	-292	-1385	0.47	D7
Corner mullion connections at base	S-3	MUL 806	40.8	Gravity	-11.5	-0.4	-0.1	0	-2	-1		
(counteract)		n=828		H. Seismic	6.7	-1.2	-20.5	-26	-800	-9		
				0.300 Gravity	3.5	-0.1	0.0	0	-1	0		
				Total	-1.4	-1.7	-20.6	-26	-803	-11	2.73	D8
Mullions at top & btm of window	S-5	MUL 806	40.8	Gravity	-11.5	-0.4	-0.1	0	-2	-1		
	TS			H. Seismic	-6.7	-1.2	-20.5	-26	-800	-9		
(additive)	8x4x1/2	n=828		0.300 Gravity	-3.5	-0.1	0.0	0	-1	0		
				Total	-21.6	-1.7	-20.6	-26	-803	-11	1.21	D9
Mullions within window span	S-5	MUL 1006	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-18		
	TS			H. Seismic	-2.3	-2.0	-8.7	35	-435	-105		
(additive)	8x4x1/2	n=1012		0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
				Total	-8.5	-2.6	-8.8	35	-437	-128	0.81	D10
Tubes at base of window	S-5	BW 1009	57.6	Gravity	-0.8	0.5	0.0	-1	0	-6		
	TS			H. Seismic	-1.0	10.0	1.1	-12	-54	-567		
(additive)	7x7 3/16	n=1008		0.300 Gravity	-0.2	0.1	0.0	0	0	-2		
				Total	-2.0	10.6	1.1	-14	-55	-574	1.23	D11
Tubes at top of window	S-5	TW 1109	72.4	Gravity	0.8	0.3	0.0	0	1	-5		
	TS			H. Seismic	1.1	5.2	-0.5	-7	34	-368		
(additive)	7x7 3/16	n=1108		0.300 Gravity	0.2	0.1	0.0	0	0	-1		
				Total	2.0	5.6	-0.6	-8	35	-374	0.79	D12

Table 48. Forces, Moments, and DCRs (SLO5, no windows, $\theta_L = 90^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 103	146.1	Gravity	-1.3	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-46.3	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=102		0.300 Gravity	-0.4	0.0	0.0	0	0	0		
				Total	-48.0	-0.1	0.0	0	0	0	1.70	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-81.9	-0.7	0.1	0	-15	-79		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-151.2	-0.7	0.1	0	-15	-80	0.61	D2
Beam at Intermediate Levels	S-2, S-4	I 210	22.6	Gravity	0.0	1.8	0.0	0	0	-69		
	W14x34			H. Seismic	0.0	7.4	0.0	0	0	-677		
(additive)	W16x40			0.300 Gravity	0.0	0.5	0.0	0	0	-21		
	W16x45	n=210		Total	0.0	9.7	0.0	0	0	-767	0.30	D3
Bracing conn at base	S-4	BR 103	-	Gravity	-1.3	-0.1	0.0	0	0	0		
- Tension				H. Seismic	46.3	0.0	0.0	0	0	0		
(counteract)		n=102		-0.300 Gravity	0.4	0.0	0.0	0	0	0		
				Total	45.4	0.0	0.0	0	0	0	0.57	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4	J 513	50.0	Gravity	0.0	-0.6	0.0	0	0	-32		
(additive)	W14x34			H. Seismic	0.0	-27.9	0.0	0	0	-712		
	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-10		
	W16x45	n=511		Total	0.0	-28.7	0.0	0	0	-754	0.41	D5
Bracing btwn Junct & Wlkwy	S-4	BR 503	102.3	Gravity	-5.0	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-45.7	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=501		0.300 Gravity	-1.5	0.0	0.0	0	0	0		
				Total	-52.2	0.0	0.0	0	0	0	1.38	D6
Beams at Base of mullions	S-2, S-3	TS 807	82.0	Gravity	0.6	1.0	0.0	0	-2	129		
(additive)	W8x35,			H. Seismic	6.1	6.7	-1.3	1	-103	547		
	W10x26			0.300 Gravity	0.2	0.3	0.0	0	-1	39		
	W16x77	n=808		Total	6.9	8.0	-1.3	1	-105	714	0.76	D7
Corner mullion connections at base	S-3	MUL 803	40.8	Gravity	-5.7	-0.1	-0.6	1	-20	-5		
(counteract)				H. Seismic	8.9	-2.9	-13.8	72	-503	-132		
		n=809		0.300 Gravity	1.7	0.0	-0.2	0	-6	-2		
				Total	4.9	-3.1	-14.5	73	-530	-139	2.13	D8
Mullions at top & btm of window	S-5	MUL 803	40.8	Gravity	-5.7	-0.1	-0.6	1	-20	-5		
(additive)	TS			H. Seismic	-8.9	-2.9	-13.8	72	-503	-132		
	8x4x1/2	n=809		0.300 Gravity	-1.7	0.0	-0.2	0	-6	-2		
				Total	-16.3	-3.1	-14.5	73	-530	-139	0.92	D9
Mullions within window span	S-5	MUL 1004	94.8	Gravity	-3.5	-0.3	0.1	0	3	-14		
(additive)	TS			H. Seismic	-2.7	-3.3	6.8	5	337	-162		
	8x4x1/2	n=1008		0.300 Gravity	-1.1	-0.1	0.0	0	1	-4		
				Total	-7.3	-3.7	6.9	5	341	-180	0.70	D10
Tubes at base of window	S-5	BW 1007	57.5	Gravity	-0.8	-0.6	0.0	0	-1	-12		
(additive)	TS			H. Seismic	-3.6	-10.7	-1.1	39	-55	-595		
	7x7 3/16	n=1008		0.300 Gravity	-0.2	-0.2	0.0	0	0	-4		
				Total	-4.6	-11.5	-1.2	40	-57	-610	1.33	D11
Tubes at top of window	S-5	TW 1105	72.4	Gravity	0.8	0.3	0.0	0	1	-6		
(additive)	TS			H. Seismic	1.5	5.4	-0.7	15	42	-369		
	7x7 3/16	n=1104		0.300 Gravity	0.2	0.1	0.0	0	0	-2		
				Total	2.5	5.8	-0.7	15	43	-376	0.81	D12

Table 49. Forces, Moments, and DCRs (SLO6, no windows, $\theta_L = 45^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 108	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-40.0	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		0.300 Gravity	-2.2	0.0	0.0	0	0	0		
				Total	-49.7	-0.1	0.0	0	0	0	1.788	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-88.3	-0.5	0.3	0	-32	-59		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-157.6	-0.5	0.3	0	-32	-60	0.651	D2
Beam at Intermediate Levels	S-2, S-4 W14x34 W16x40	I 223	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
(additive)	W16x45	n=219		H. Seismic	0.0	-25.1	0.0	0	0	-624		
				0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
				Total	0.0	-26.2	0.0	0	0	-697	0.366	D3
Bracing conn at base	S-4	BR 108	-	Gravity	-7.4	-0.1	0.0	0	0	0		
- Tension				H. Seismic	40.0	0.0	0.0	0	0	0		
(counteract)		n=105		-0.300 Gravity	2.2	0.0	0.0	0	0	0		
				Total	34.8	-0.1	0.0	0	0	0	0.44	D4
Beam at Junct. level (El. 29'-8")	S-2, S-4 W14x34 W16x40	J 527	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
(additive)	W16x45	n=523		H. Seismic	0.0	-19.2	0.0	0	0	-489		
				0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
				Total	0.0	-21.9	0.0	0	0	-589	0.31	D5
Bracing btwn Junct & Wlkwy	S-4	BR 508	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-44.6	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		0.300 Gravity	-2.7	0.0	0.0	0	0	0		
				Total	-56.4	-0.1	0.0	0	0	0	1.69	D6
Beams at Base of mullions	S-2, S-3 W8x35, W10x26	TS 830	80.6	Gravity	0.8	-1.1	0.0	0	-3	132		
(additive)	W16x77	n=824		H. Seismic	10.3	-5.5	0.4	1	-34	442		
				0.300 Gravity	0.2	-0.3	0.0	0	-1	40		
				Total	11.3	-7.0	0.5	1	-39	613	0.79	D7
Corner mullion connections at base	S-3	MUL 805	40.8	Gravity	-6.5	-0.1	0.7	-2	24	-7		
(counteract)		n=825		H. Seismic	13.6	-2.1	13.0	-54	490	-130		
				0.300 Gravity	2.0	0.0	0.2	-1	7	-2		
				Total	9.0	-2.2	14.0	-57	522	-139	2.12	D8
Mullions at top & btm of window	S-5	MUL 805	40.8	Gravity	-6.5	-0.1	0.7	-2	24	-7		
(additive)	TS			H. Seismic	-13.6	-2.1	13.0	-54	490	-130		
	8x4x1/2	n=825		0.300 Gravity	-2.0	0.0	0.2	-1	7	-2		
				Total	-22.1	-2.2	14.0	-57	522	-139	0.92	D9
Mullions within window span	S-5	MUL 1006	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-16		
(additive)	TS			H. Seismic	-4.9	-5.2	-5.9	6	-294	-235		
	8x4x1/2	n=1012		0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
				Total	-11.0	-5.7	-5.9	6	-297	-256	0.72	D10
Tubes at base of window	S-5	BW 1011	57.6	Gravity	-0.8	-0.7	0.0	0	0	-18		
(additive)	TS			H. Seismic	-2.0	-8.5	-0.9	-20	-47	-476		
	7x7 3/16	n=1012		0.300 Gravity	-0.2	-0.2	0.0	0	0	-5		
				Total	-3.1	-9.4	-0.9	-21	-47	-499	1.08	D11
Tubes at top of window	S-5	TW 1109	72.4	Gravity	0.8	0.3	0.0	0	1	-5		
(additive)	TS			H. Seismic	3.0	4.7	-0.6	13	34	-318		
	7x7 3/16	n=1108		0.300 Gravity	0.2	0.1	0.0	0	0	-2		
				Total	4.0	5.1	-0.6	13	35	-325	0.70	D12

Table 50. Forces, Moments, and DCRs (SLO7, no windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at Base	S-4	BR 108	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
- Buckling				H. Seismic	-52.5	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		0.300 Gravity	-2.2	0.0	0.0	0	0	0		
				Total	-62.2	-0.1	0.0	0	0	0	2.58	D1
Column at Base	S-4	C 104	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
				H. Seismic	-96.1	-0.3	0.4	0	-42	-33		
(additive)	W8x31	n=104		0.300 Gravity	-16.0	0.0	0.0	0	0	0		
				Total	-165.4	-0.3	0.4	0	-42	-34	0.68	D2
Beam at Intermediate Levels	S-2, S-4 W14x34	I 223	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
	W16x40			H. Seismic	0.0	-33.0	0.0	0	0	-824		
(additive)	W16x45	n=219		0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
				Total	0.0	-34.0	0.0	0	0	-897	0.47	D3
Bracing conn at base	S-4	BR 108	-	Gravity	-7.4	-0.1	0.0	0	0	0		
- Tension				H. Seismic	52.5	0.0	0.0	0	0	0		
(counteract)		n=105		-0.300 Gravity	2.2	0.0	0.0	0	0	0		
				Total	47.3	-0.1	0.0	0	0	0	0.60	D4
Beam at Junct. level (EI. 29'-8")	S-2, S-4 W14x34	J 527	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
	W16x40			H. Seismic	0.0	-25.6	0.0	0	0	-653		
(additive)	W16x45	n=523		0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
				Total	0.0	-28.4	0.0	0	0	-754	0.40	D5
Bracing btwn Junct & Wlkwy	S-4	BR 508	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
- Buckling				H. Seismic	-59.5	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		0.300 Gravity	-2.7	0.0	0.0	0	0	0		
				Total	-71.3	-0.1	0.0	0	0	0	2.33	D6
Beams at Base of mullions	S-2, S-3 W8x35, W10x26	TS 833	43.6	Gravity	-3.4	-7.3	-0.3	0	-13	-65		
				H. Seismic	-7.5	-40.5	-5.2	-5	-237	-1263		
(additive)	W16x77	n=827		0.300 Gravity	-1.0	-2.2	-0.1	0	-4	-19		
				Total	-11.9	-50.0	-5.5	-5	-253	-1347	0.43	D7
Corner mullion connections at base (counteract)	S-3	MUL 806	40.8	Gravity	-11.5	-0.4	-0.1	0	-3	-1		
				H. Seismic	6.5	-1.1	-18.2	-17	-728	-10		
				0.300 Gravity	3.4	-0.1	0.0	0	-1	0		
		n=828		Total	-1.5	-1.6	-18.3	-17	-731	-11	2.49	D8
Mullions at top & btm of window	S-5	MUL 806	40.8	Gravity	-11.5	-0.4	-0.1	0	-3	-1		
				H. Seismic	-6.5	-1.1	-18.2	-17	-728	-10		
(additive)	8x4x1/2	n=828		0.300 Gravity	-3.4	-0.1	0.0	0	-1	0		
				Total	-21.4	-1.6	-18.3	-17	-731	-11	1.11	D9
Mullions within window span	S-5	MUL 1006	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-16		
				H. Seismic	-2.1	-2.2	-8.0	5	-400	-103		
(additive)	8x4x1/2	n=1012		0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
				Total	-8.2	-2.7	-8.0	5	-402	-124	0.75	D10
Tubes at base of window	S-5	BW 1010	57.6	Gravity	-0.8	-0.4	0.0	0	1	-3		
				H. Seismic	-2.2	-10.0	0.7	26	36	-577		
(additive)	7x7 3/16	n=1009		0.300 Gravity	-0.2	-0.1	0.0	0	0	-1		
				Total	-3.2	-10.6	0.7	27	36	-581	1.22	D11
Tubes at top of window	S-5	TW 1112	72.4	Gravity	0.8	0.4	0.0	0	-1	-12		
				H. Seismic	2.1	5.6	0.7	-18	-44	-362		
(additive)	7x7 3/16	n=1112		0.300 Gravity	0.2	0.1	0.0	0	0	-4		
				Total	3.1	6.1	0.7	-18	-45	-377	0.82	D12

Table 51. Mullion Connections at Their Bases (SL07, no windows, $\theta_L = 0^\circ$, $S_{DS} = 1.00g$).

[illegible]

7 Summary

This report presents the detailed seismic evaluation of the FAA ATCTs at the locations given in Table 1. These include Type L towers located in Salinas, San Carlos, and Palo Alto, CA. A unique eccentrically braced steel frame tower located in San Luis Obispo was also evaluated. Each of these towers was evaluated based on the maximum considered earthquake defined by 1997 NEHRP Recommended Provisions (FEMA 302). Both Type L and the San Luis Obispo towers were evaluated based on several directions of loading and an extreme assumption that the cab windows will not fail and will work as fully effective shear walls.

Type L ATCTs

The San Carlos ATCT is the most critical Type L tower, due to excessive deflections in the tower cab. These deflections were due to large rotations of supporting members at the shaft roof. The cab columns (corner mullions) connection base plates were also overstressed as indicated by very large DCRs. Hinges would form at the base of each mullion due to base plate bending failure, causing a collapse mechanism at very low seismic motions.

An upgrade approach was developed and demonstrated that reduces deflections to acceptable levels and protects the vulnerable connections. This upgrade consists of welding deep structural tubing members to the base of each corner mullion in a pentagon configuration as shown in Figure 11. The mullions themselves were also stiffened and strengthened by welding 5 in. x 1.5 in. plates on both faces of the mullions.

San Luis Obispo ATCT

The shaft braces were the most critically stressed components in the San Luis Obispo shafts. These would buckle at several floor levels. However, when the braces were assumed to be tension only members, they had adequate capacity. There could be slight yielding of these braces, but this would be very limited and deflections would be kept within acceptable levels.

Deflection in the tower cab could be large, but within acceptable levels. The most vulnerable cab component is the connection of the corner mullions to their base plates. This is due to shear failure of the fillet welds at this connection. However, serious damage to these connections should be prevented by redistribution of forces to other mullions and other building components. Therefore, the San Luis Obispo tower passed this evaluation by meeting the life-safety requirements.

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